PB83-259739

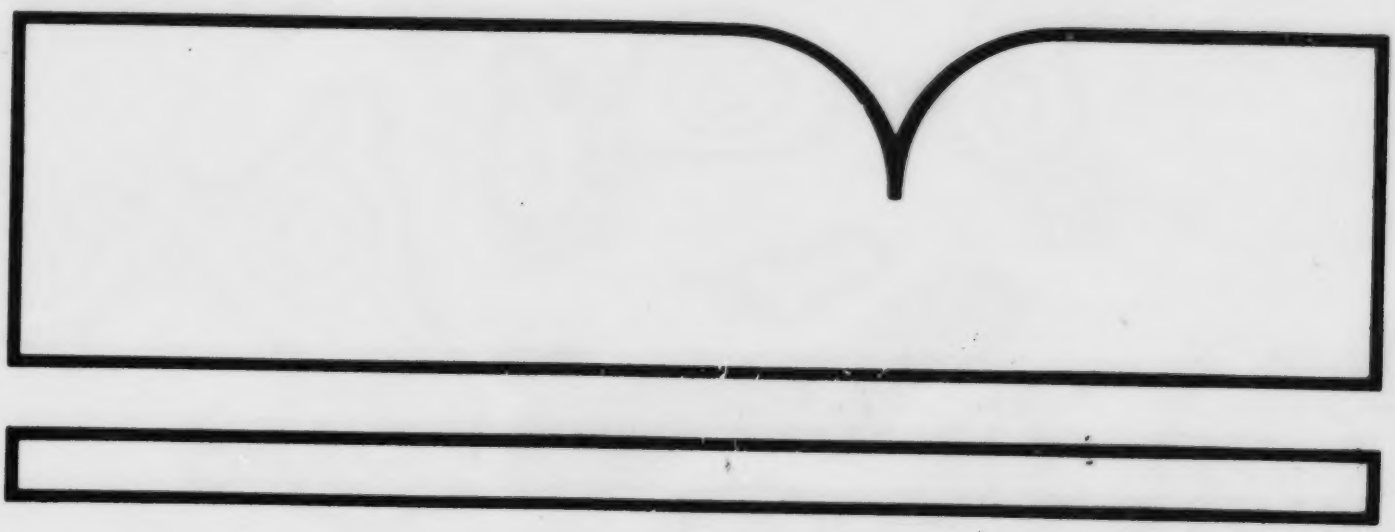
Value of Flow Calibration for Decision Making in Infiltration/Inflow Studies

Environmental Design and Planning, Inc.
Boston, MA

Prepared for

Municipal Environmental Research Lab.
Cincinnati, OH

Sep 83



U.S. Department of Commerce
National Technical Information Service
NTIS

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO.	2.	3. RECIPIENT'S ACCESSION NO.
EPA-600/2-83-087		

4. TITLE AND SUBTITLE
THE VALUE OF FLOW CALIBRATION FOR DECISION MAKING IN INFILTRATION/INFLOW STUDIES

5. REPORT DATE
September 1983

6. PERFORMING ORGANIZATION CODE

7. AUTHOR(S)
William C. Pisano, David S. Watson, and Gerald L. Aronson

8. PERFORMING ORGANIZATION REPORT NO.

9. PERFORMING ORGANIZATION NAME AND ADDRESS
Environmental Design & Planning, Inc.
19 Fordham Road
Allston, Massachusetts 02134

10. PROGRAM ELEMENT NO.

11. ~~CONTRACT~~/GRANT NO.
R-804578

12. SPONSORING AGENCY NAME AND ADDRESS
Municipal Environmental Research Laboratory - Cin., OH
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

13. TYPE OF REPORT AND PERIOD COVERED

14. SPONSORING AGENCY CODE
EPA/600/14

15. SUPPLEMENTARY NOTES Project Officer: Richard Field, Chief, Storm and Combined Sewer Program; Comm. (201) 321-6674; FTS 340-6674.

16. ABSTRACT

This report deals with the value of flow calibration for decision making in sewer system studies. Primary data used in this investigation were the field measurement results from a prior infiltration/inflow study conducted for the Metropolitan District Commission (MDC), Boston, covering portions of West Roxbury-Newton-Brookline-Dedham in the Boston metropolitan area. The results are applicable for any sewer system flow measurement program.

It was determined in the prior study that the conventional use of Manning's equation for relating stage records collected by manual and automated recorders, without primary flow calibration, would grossly misestimate discharge conditions over all flow regimes. This problem translates into erroneous cost conclusions regarding treatment, transport and rehabilitation recommendations. A number of primary discharge measurements at key manholes in the study area were taken during both dry and wet weather flow conditions. The prior study did not pursue the notion of the value of primary gaging as a means to reduce economic regret or the cost of erroneous decision making.

The present study included a variety of "what if" scenarios for critical key manholes as to the amount of information available to develop stage/flow calibration curves. Alternative stage/flow curves were used as the basis for costing the resulting rehabilitation/treatment programs. In short, the effort investigated the marginal problem of increasing gaging costs, while potentially reducing the uncertainty of erroneous recommendations. The findings indicate that the additional care and the costs involved in primary flow calibration of stage/flow curves for sewer system evaluations are well worth the effort, and, in many instances, could save the government valuable resources.

This report was submitted in partial fulfillment of Grant No. R-804578 by Northeastern University and Environmental Design & Planning, Inc., under the sponsorship of the U.S. Environmental Protection Agency. This report covers the period of May 1, 1978 to May 30, 1979. Work was completed as of April 1980.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Flow measurement*, Flowmeters, Liquid flow, Open channel flow, Pipe flow, Stream flow, Storm sewers, Combined sewers, Overflows, Manholes, Outfall sewers, Sanitary engineering, Urban areas, Water pollution, Flow, Sewers	Water pollution control, Flow calibration	13B

18. DISTRIBUTION STATEMENT	19. SECURITY CLASS *(This Report)*	21. NO. OF PAGES
RELEASE TO PUBLIC	UNCLASSIFIED	75
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	22. PRICE

EPA Form 2220-1 (Rev. 4-77) PREVIOUS EDITION IS OBSOLETE

PB83-259739

EPA-600/2-83-087
September 1983

THE VALUE OF FLOW CALIBRATION FOR DECISION MAKING IN INFILTRATION/INFLOW STUDIES

by

William C. Pisano
David S. Watson
Gerald L. Aronson

Environmental Design & Planning, Inc.
Allston (Boston), MA 02134

Grant No. R-804578

Project Officers:

Richard Field
Robert Turkeltaub
Storm and Combined Sewer Program
Wastewater Research Division
Municipal Environmental Research Laboratory (Cincinnati)
Edison, New Jersey 08837

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U.S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA 22161

DISCLAIMER

Although the information described in this article has been funded wholly or in part by the United States Environmental Protection Agency through assistance agreement number R804578 to Environmental Design & Planning, Inc., it has not been subjected to the Agency's required peer and administrative review and therefore does not necessarily reflect the views of the Agency and no official endorsement should be inferred. Mention of trade names or commercial products does not constitute endorsement or recommendation for use.

FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution, and it involves defining the problem, measuring its impact and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health and aesthetic effects of pollution. This publication is one of the products of that research, a most vital communications link between the researcher and the user community.

The deleterious effects of storm sewer discharges and combined sewer overflows upon the nation's waterways have become of increasing concern in recent times. Efforts to alleviate the problem depend upon the development of integrated technologies involving non-structural best management practices with structural storage and treatment concepts.

This report represents the results of an analysis of the findings of an infiltration/inflow study covering portions of West Roxbury, Newton, Brookline and Dedham in the Boston metropolitan area, which was recently completed by Environmental Design & Planning, Inc. for the Metropolitan District Commission of Boston. It was determined that the conventional use of Manning's equation for relating stage records collected by manual and automated recorders, without primary flow calibration, would grossly misestimate discharge conditions over all flow regimes. This problem translates into erroneous cost conclusions regarding treatment, transport and rehabilitation recommendations. This report includes a variety of "what if" scenarios for critical key manholes, as to the amount of information available to develop stage/flow calibration curves. Alternative stage/discharge curves were used as the basis for costing the resulting rehabilitation/treatment

programs. This study analyzes the marginal problem of increasing gaging costs while potentially reducing the uncertainty of erroneous recommendations, resulting in better use of Federal government and local resources.

Francis T. Mayo
Director
Municipal Environmental Research
Laboratory

BEGIN FIRST LINE OF TEXT HERE
TYPING GUIDE SHEET
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD BEGIN SECTIONS HERE

ABSTRACT

This report deals with the value of flow calibration for decision making in sewer system studies. Primary data used in this investigation were the field measurement results from a prior infiltration/inflow study conducted for the Metropolitan District Commission (MDC), Boston, covering portions of West Roxbury-Newton-Brookline-Dedham in the Boston metropolitan area. The results are applicable for any sewer system flow measurement program.

It was determined in the prior study that the conventional use of Manning's equation for relating stage records collected by manual and automated recorders, without primary flow calibration, would grossly misestimate discharge conditions over all flow regimes. This problem translates into erroneous cost conclusions regarding treatment, transport and rehabilitation recommendations. A number of primary discharge measurements at key manholes in the study area were taken during both dry and wet weather flow conditions. The prior study did not pursue the notion of the value of primary gaging as a means to reduce economic regret or the cost of erroneous decision-making.

The present study included a variety of "what if" scenarios for critical key manholes as to the amount of information available to develop stage/flow calibration curves. Alternative stage/flow curves were used as the basis for costing the resulting rehabilitation/treatment programs. In short, the effort investigated the marginal problem of increasing gaging costs, while potentially reducing the uncertainty of erroneous recommendations. The findings indicate that the additional care and the costs involved in primary flow calibration of stage/flow curves for sewer system evaluations are well worth the effort, and, in many instances, could save the government valuable resources.

This report was submitted in partial fulfillment of Grant No. R804578 by Northeastern University and Environmental Design & Planning, Inc., under the sponsorship of the U.S. Environmental Protection Agency. This report covers the period of May 1, 1978 to May 30, 1979. Work was completed as of April, 1980.

BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA, OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
3/8"

PAGE NUMBER
EPA-287 (Cin.)
(4-76)

CONTENTS

Disclaimer ii
Foreword iii
Abstract v
Figures vii
Tables viii
Abbreviations and Symbols ix
Acknowledgement x

1. Introduction 1
2. Conclusions 6
3. Recommendations 8
4. Overview of Prior Infiltration/Inflow Study 10
5. Theoretical Details for Least Squares Approaches 20
6. Discharge Computations 37
7. Alternative Rehabilitation Program Costs 51

References 64

FIGURES

Number		Page
1	Schematic of R&D Effort	5
2	Study Area Location Map	11
3	Schematic Flow Diagram of Key Manhole Areas	12
4	Schematic Diagram of MDC Neponset Valley Sewer	15
5	Alternative Stage/Discharge Curves - Area 2	29
6	Alternative Stage/Discharge Curves - Area 25	30
7	Alternative Stage/Discharge Curves - Area 28	31
8	Alternative Stage/Discharge Curves - Area 32	32
9	Alternative Stage/Discharge Curves - Area 41	33
10	Stage/Discharge Curves for Area 1	38
11	Stage/Discharge Curves for Area 4	39
12	Stage/Discharge Curves for Area 20	40
13	Stage/Discharge Curves for Area 23	41
14	Stage/Discharge Curves for Area 43	42
15	Typical Weekly Flow Gaging Results - Area 20	43

TABLES

Number		Page
1	Pipe Lengths Within Each Analysis Area	13
2	Acres Contained Within Each Analysis Area	14
3	Summary of Field Discharge Calibration Program Data	18
4	Tabulation of Manning's Roughness Coefficient Versus Depth of Flow	22
5	Sample Calculations of Least Squares Procedure	25
6	Sensitivity Analysis of Least Squares Procedures	26
7	Comparison of Discharges Computed Using Various Stage/Discharge Formulations	28
8	Ratios of Flow Computed Using Pipe Slope from Drawings with Flows Computed from Least Square Fit of Calibration Data	34
9	Estimation of Elevation Error Causing Average Error Between Curves 1 & 3 (Figures 5-9) for Areas 2, 25, 28, 32 and 41	35
10	Summary of Contributory Discharges to MDC Interceptor: Method 1.	45
11	Summary of Contributory Discharges to MDC Interceptor: Method 2.	46
12	Summary of Contributory Discharges to MDC Interceptor: Method 3.	47
13	Overall Comparison of Discharge for Different Flow Conditions Computed by Three Stage/Discharge Methods	49
14	Relative Ratios of Discharge for Different Flow Conditions Computed by Three Stage/Discharge Methods	50
15	Overview of Rehabilitation Program - Method 1	53
16	Overview of Rehabilitation Program - Method 2	54
17	Overview of Rehabilitation Program - Method 3	55
18	Infiltration Rehabilitation Program Details	57
19	Relief Sewer Construction Program	59
20	Present Value Treatment Costs for Extraneous Wet Weather Discharges	60
21	Comparison of Overall Infiltration/Inflow Correction Cost Programs	61
22	Overview of Control Plans Developed Using Different Methods of Flow Estimation	63

LIST OF ABBREVIATIONS AND SYMBOLS

ABBREVIATIONS

cfs	- cubic feet per second
cm	- centimeter
dwf	- dry weather sewage flow
fps	- feet per second
ft	- feet
gal	- gallons
gpm	- gallons per minute
ha	- hectare
in.	- inch
l/s	- liters per second
kg	- kilogram
kg/sq m	- kilogram per square meter
m	- meter
m/s	- meter per second
mi	- mile
ols	- ordinary least squares
O&M	- operation and maintenance
vcp	- vitrified clay pipe
wls	- weighted least squares

SYMBOLS

a	- Flow cross-sectional area (ft^2)
a_i	- Flow cross-sectional area, ith measurement
D	- Pipe diameter of sewer segment
$F(h_i/D)$	- Function of flow depth i relative to diameter, D
h	- Water depth (ft)
h_i	- Water depth, ith measurement
$K_i(h_i/D)$	- Product of $(1.49\ a_i\ r_i^{2/3})/n_i$
m	- Summation index
n	- Manning's roughness coefficient
n_f	- Manning's roughness coefficient at full pipe
n_i	- Manning's roughness coefficient for ith measurement
$O(\ )$	- Objective function
Q	- Flow rate (cfs)
Q_i	- Flow rate corresponding to ith measurement
r	- Hydraulic radius (ft)
r_i	- Hydraulic radius of ith measurement
S	- Energy slope
S_o	- Pipe slope
$\hat{S}$	- Estimated energy slope

NOTATION

Σ	- Summation

ACKNOWLEDGEMENTS

The successful completion of this report was dependent on the cooperation and assistance of a number of individuals and organizations. We are indebted particularly to Richard Field, Chief of the Storm and Combined Sewer Section, Municipal Environmental Research Laboratory Cincinnati, EPA, Edison, New Jersey, Richard Traver and Robert Turkeltaub, Staff Engineers of the Storm and Combined Sewer Section, for their guidance and review of the work.

This work was conducted under the supervision and direction of Dr. William C. Pisano, President, Environmental Design & Planning, Inc.(EDP). Mr. David S. Watson, Director of Field Services, EDP, prepared the data base for the analyses. Mr. Gerald L. Aronson was the project manager for the infiltration/inflow study in metropolitan Boston, which was used as the case study in this investigation.

SECTION 1

INTRODUCTION

1.1 Foreword

Accurate determination of dry and wet weather sewerage system flow characteristics is an important step in the establishment of infiltration and/or inflow rehabilitation requirements as defined by the FWPCA Amendments, to PL 92-500. Flow gaging in small diameter collection systems meeting the EPA key manhole/mileage constraint is difficult to accomplish in practice. Inaccurate gaging in some instances, particularly for borderline decision situations, may lead to erroneous recommendations for future actions resulting in the inefficient allocation of local and/or federal resources.

Flow determinations are often performed in infiltration/inflow studies using "dipstick" instantaneous spot procedures to obtain stage levels, and then converted into discharge using any one of a variety of different procedures such as Manning's, Chezy and/or Kutter's equation, together with various hydraulic input parameters (size, shape, length and slope) taken from plan and profile "as-built" drawings. Roughness coefficients are usually estimated by inspection of the condition of the test segment and age since construction. In other situations, automatic stage recording devices are installed to obtain a long-time series of stage records. Most often, these records are then converted to discharge quantities using similar procedures without any primary flow determination to calibrate any of the procedures for relating stage to discharge. For large diameter pipes with adequate discharge, say flowing half-full, the error of this approach is probably acceptable. The opposite situation is more likely to be true for smaller diameter pipes (or large combined sewer laterals) with typically several inches of flow.

1.2 Discussion of Flow Determination Methods

Accurate measurement of either dry or wet weather flow discharge characteristics in a sewerage system is not a straightforward field engineering task. It is a difficult process beset by constantly changing problems. For example, sediment accumulations erratically increase values of roughness coefficients nominally determined for clean pipe conditions, as well as change the cross-sectional area of flow. Accurate sediment depth determination is extremely difficult due to the compressibility of highly organic deposits. Grit and debris tend to impair velocity determinations and thereby can bias discharge computations. Backwater problems elevate depths of flow as well as change velocity profiles.

Two different measurement procedures can be used: direct methods where both depth of flow and velocity measurements are made, and indirect methods where only depth of flow is recorded, and then converted to discharge using some predictive relationship. Direct measurement approaches can provide accurate determinations of flow for given instants of time, but are not routinely used in gaging studies. Secondary, or indirect, methods provide a less accurate flow estimate but are commonly used in infiltration/inflow studies because of their simplicity and low cost. In many studies no direct measurements are made to corroborate or calibrate indirect methods.

Direct methods measure pipe velocity at a given instant of time, and, together with an accurate determination of effective flowing cross-section, render an estimate of flow. Current meters and dye/tracer methods are commonly used for such determinations. Recent technological improvements in trace fluorometric equipment can provide long-term flow determinations, but the actual job experiences using these devices are too few to warrant general conclusions. Recent equipment operating on direct electromagnetic and ultrasonic principles are reasonably priced, and provide reliable instantaneous measurements. Technical problems are often encountered using direct methods, and include: a) visual sighting of dispersed first peak versus centroid estimation using dye tracer methods; b) depth of flow determination at high velocities; c) realistic estimates of sediment and debris; and, d) accurate estimates of average cross-sectional velocity at either low or high discharge rates, that is, less than 0.025 fps (0.008 m/s) or greater than 10 fps (3.33 m/s).

Indirect methods usually entail some form of sensing liquid level in a flowing pipe, either performed manually or using an automated continuous recording device. Weirs or flumes are occasionally used to create critical flow conditions for flow determination.(Serious biasing problems can result by using these devices due to both sediment and floating debris accumulations directly behind the weir. This condition is especially a critical problem in gaging small diameter sanitary sewers with several inches of flow in the collection systems.) The recorded stages are then transformed into discharge values using some predictive formalism that relates discharge to physical hydraulic parameters of the pipe segment. Commonly used relationships include Manning's, Chezy and/or Kutter's formula for open-channel flow computations. All of these relationships required that several physical parameters such as pipe slope, cross-sectional dimensions and pipe roughness be known. Most often, flows are computed using Manning's equation, using slopes obtained from design or as-built drawings and a constant friction factor from the literature. Such applications of Manning's equation may lead to flow estimations which are substantially different from the true flows. The uncertainty of error in the stage-to-flow translation step inherent in all indirect methods can be substantially reduced using direct flow measurement techniques to establish calibrated stage/discharge rating curves for important key manholes.

One approach used in some studies is to obtain primary measurements at several flow depths (most often at dwf conditions) and then use Manning's equation (or an equivalent) in dimensionless form, to estimate full pipe discharge conditions so that the upper end of the stage/discharge curve can

be established. This method is very reasonable provided that the calibration points are made without measurement error (which is extremely unlikely) and that the variation of roughness with depth is known with reasonable certainty. The presence of sediment beds complicates the numerical simplicity of this direct approach. It is recognized that this approach may be acceptable in some instances, but the method skirts the inherent issue of measurement error and the attendant uncertainties and approximations associated with Manning's equation. Manning's equation is at best an approximate model of open channel flow requiring a number of requiste conditions and assumptions. An alternative methodology is presented in this report consisting of simple statistical least squares curve fitting techniques which squarely meet both the measurement error problem and the inherent limitation of using a simple model (Manning's equation) to represent sometimes complicated phenomena.

1.3 Purpose of Study

This study investigates, by way of a case example in Eastern Massachusetts, the informational value of differing degrees of careful versus "conventional" gaging procedures in relationship to cost differences in recommended plans or actions. The central idea is to show how careful gaging in infiltration/inflow work can potentially reduce the possibility of erroneous recommendations.

1.4 Executive Overview

A schematic of the overall project effort, including prior work, is depicted in Figure 1. In the prior study (1), sewage flow depths at key manholes throughout a 3000-acre sewerage system were measured during both dry and wet weather conditions including instantaneous and continuous measurements. Discharge estimates at a number of locations using direct measurement procedures had also been obtained. A single stage/discharge method had been used in the prior study to compute dry and wet weather flows for the purpose of preparing the cost effectiveness analysis of sewer system rehabilatation needs. In addition, alternative stage/discharge method had been examined at a representative number of key manholes. These results are reported in this study. In Step A of this study, alternative stage/discharge relations were derived for all key manholes using different assumptions. Alternative estimates of flow conditions throughout the sewerage system for both dry and wet weather conditions are computed in Step B, using the alternative stage discharge relationships derived in Step A and the raw flow depth information from the prior study. Various sewer system rehabilitation programs which attain a similar degree of extraneous flow reduction are proposed in Step C. Complete analyses of the required sewer system upgrade are prepared and costed for each set of assumed flow conditions. Finally, in Step D, an overall assessment is made showing the economic benefits of measurements versus the implied costs of the alternative rehabilitation programs.

1.5 Report Outline

A review of the prior study field program results is presented in Section 4. Theoretical details of various least squares fitting procedures for using primary flow information to develop stage/discharge curves

are presented in Section 5. Alternative stage/discharge curves for various key manholes in the study area and various estimates of overall sewerage system flows using these alternative relationships are presented in Section 6. The resulting sewer system rehabilitation programs and their costs are given in Section 7. A discussion of the benefits of carefully utilizing primary flow information in sewerage system evaluation is also presented in Section 7.

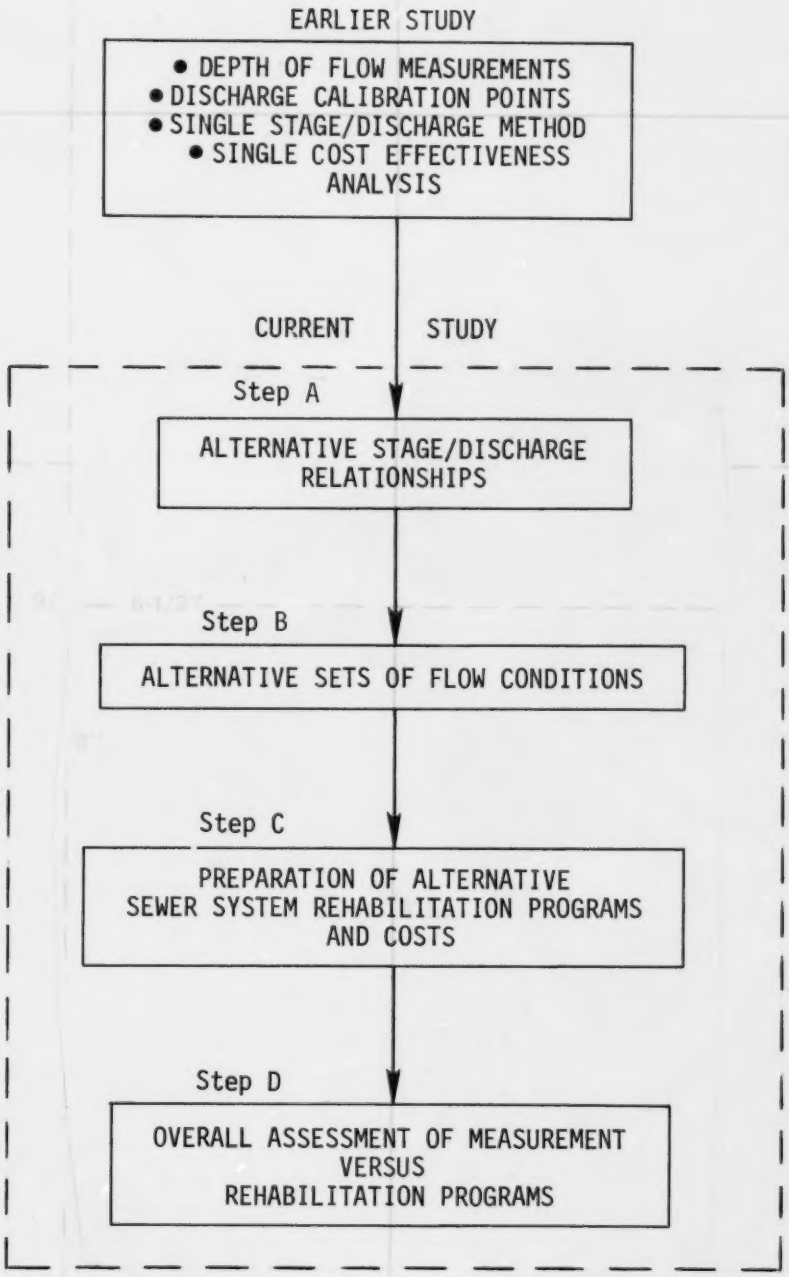


Figure 1. Schematic of Project Research and Development.

SECTION 2

CONCLUSIONS

1. Measurement of flow in open channel sewer pipes is most often conducted by measuring flow depth and using some formulation, usually Manning's equation, to convert to actual flow. In the case of infiltration/inflow studies, the flow data so generated is then used to develop corrective infiltration/inflow control programs. Straightforward application of Manning's equation using plan & profile slopes and a fixed roughness coefficient without calibration data can result in gross misestimation of discharge. Utilization of misestimated discharge levels in engineering analysis can lead to erroneous control plan recommendations and consequently, inefficient utilization of federal and local resources.

2. Several statistical methods of developing stage/discharge relationships are presented in which Manning's equation with a variable roughness coefficient is curve-fitted by least squares methods to calibration data obtained by direct field measurements. A surrogate value of energy slope is determined that minimizes the sum of squares of field discharge information about a curve defined by Manning's equation. The procedure makes the best use of limited depth of flow/discharge measurements in developing the overall rating curve. Alternative fitting procedures are developed using ordinary least squares and weighted least squares approaches.

3. Depth of flow information collected at 45 key manholes during a prior infiltration/inflow study(1) in the Boston metropolitan area were used as the data base in a study comparing discharge estimation by three different methods. The three alternative procedures for developing stage/rating curves included: a) application of weighted least squares procedures to Manning's equation using a variable roughness coefficient; b) application of Manning's equation using plan & profile slope and a variable roughness coefficient formulation; and, c) application of Manning's equation using plan & profile slope with a fixed roughness coefficient. The comparison is meant to show the differences between the least squares procedure and methods commonly used in practice. Stage/rating curves were developed at each key manhole for each method of discharge estimation. Estimates of peak dry weather flow, infiltration and inflow were prepared for each key manhole measurement location using 550 days of continuous depth information at 21 locations measured by automatic level sensing recorders and 320 spot flow determinations performed during dry weather, inflow and infiltration conditions. Overall system flow estimates using methods b and c misestimated (overestimated) discharge in comparison to those computed by method a under all flow conditions by as much as 51%. Positive differences between methods a versus b and c for individual locations were as high as 200%. For several measure-

ment locations the reverse was true since the estimated discharge levels were much higher than those derived using methods b and c. Flow estimates prepared using method b (variable roughness coefficient and plan & profile slope) were significantly less than those computed using method c (fixed roughness coefficient and plan & profile slope), particularly at low flow (dwf) conditions. The flow estimation differences between these two methods decreased and in some cases converged, for (near full pipe/full pipe) wet weather flow conditions as the roughness coefficient in the variable formulation (method b) approached the assumed fixed roughness value (method c).

4. It was noted that the least squares approach using variable roughness coefficient fitted observed data better than a least squares approach using a fixed roughness coefficient. These results suggest that Manning's equation is more capable of reproducing the physical phenomena of flow in pipes, if n is considered as a function of depth of flow.

5. The three different sets of peak dry weather flow, infiltration and inflow for the 45 key manholes were then used as raw information in an analysis to determine technically equivalent least-cost infiltration/inflow control programs. The controls consisted of infiltration rehabilitation and inflow remedial programs in areas tributary to an interceptor undergoing backwater and surcharge conditions together with new construction of relief (increased conveyance) segments along the existing interceptor. The aim of the effort was to investigate how the mix of final recommended controls and their associated costs would change using flows computed by the different flow estimation schemes. Identical criteria were employed in determining the level and need for each type of control for the three different methods. Present value costs were computed for each plan and included infiltration rehabilitation, remedial inflow corrections, construction of new relief segments along the existing interceptor and treatment costs of extraneous wet weather discharges. The overall control program present value costs for the three methods of flow estimation equalled $3.16 million, $3.58 million and $6.13 million, respectively. The mix of recommended controls were substantially different for the three plans. The engineering cost involved in performing the field calibration discharge measurements together with performing the least squares calculations equalled about $13,500 (from the prior study). Field calibration accounted for about 90% of the field additional effort. The least squares computations can be quickly performed using typical engineering office desk calculators.

6. The benefit of carefully preparing stage/discharge rating curves in infiltration/inflow studies by performing primary measurements and making best use of this information utilizing curve fitting procedures is significant. This conclusion obviously holds true for any sewerage system management study, particularly combined sewer overflow control plans.

SECTION 3

RECOMMENDATIONS

1. Primary flow calibration points should be used to establish stage/discharge curves since sole reliance on Manning's equation using unverified plan & profile pipe slopes can result in misestimated discharge from stage information.

2. Variable roughness coefficient in Manning's equation should be used in lieu of a fixed setting of the roughness parameter since more realistic estimates of discharge, particularly for low depths of flow, can be obtained.

3. The recommended procedures developed in this report for preparing stage/discharge rating curves are to perform primary discharge measurements and then least-squares fit the data about Manning's equation utilizing a variable roughness coefficient. The procedure requires an assumption of roughness at full pipe conditions. The recommended procedure finds a surrogate energy slope that embeds all the uncertainty regarding plan & profile slope, roughness coefficient and the requisite conditions for using Manning's equation. The procedure best utilizes the available data in an efficient manner.

4. This report does not address the issue of how many calibration points and/or the requirement of obtaining measurements at both high and low flow regimes to produce a stable curve. Obviously flow calibration points at low, mid and high flows will necessarily result in a fitted curve that is reliable at all depths of flow. It remains an open question whether high flow measurements are necessary since they are extremely difficult and expensive to routinely obtain in practice. Stated in another way, can a reliable curve for high flow regimes be prepared using low and/or mid level depths of flow. This is a critical issue since the degree of infiltration/inflow control is directly dependent on high flow estimates. A further study should be conducted to address this issue.

5. This report does not address the sensitivity issue of using Manning's equation to approximate possible nonsteady state flow regimes which can occur during high inflow conditions. It is assumed in this report that the statistical approach would embed within the steady state Manning's formulation the errors associated with approximately non-steady conditions. This problem should be further investigated.

TYPING GUIDE SHEET
FIRST LINE OF TEXT HERE
CENTER OF PAGE
TOP OF IMAGE AREA
DROPPED HEAD. BEGIN SECTIONS HERE
6 1/2"
BEGIN LAST LINE OF TEXT
BOTTOM OF IMAGE AREA. OUTSIDE DIMENSION FOR TABLES AND ILLUSTRATIONS
1/2"
PAGE NUMBER
EPA-287 (Gen.) (4-76)

6. Another critical analysis investigation of an infiltration inflow study should be performed to further examine the reliability of the various flow estimation procedures used in practice. The analysis should focus on the degree to which technical decisions, recommendations and ultimately costs, are influenced by the method(s) in which raw field engineering data are processed to produce information for engineering cost-effectiveness analyses.

SECTION 4

OVERVIEW OF PRIOR INFILTRATION/INFLOW STUDY

4.1 Foreword

A general review of the field engineering activities conducted during the prior infiltration/inflow study is presented in this Section. Other portions of the prior study are presented in Sections 5, 6, and 7. Various stage discharge relationships developed during the prior study are discussed in Section 5. Sewerage system discharge estimates along the MDC Interceptor for various flow conditions are discussed in Section 6. Rehabilitation program costs are given in Section 7.

A general description of the study area is presented in Section 4.2. Purposes and critical issues of the project are highlighted in Section 4.3. Results of the flow gaging and field inspection programs are discussed in Section 4.4. Details of the measurement calibration program conducted for most of the key manhole gaging points are discussed in Section 4.5.

4.2 Description of Study Area

The general location of the study area is shown in Figure 2. The service area encompasses roughly 3000 acres (1214 ha) with a total of 74 mi (119 km) of separate sanitary sewerage systems. The sewerage system was divided into 45 key manhole areas for the infiltration/inflow analysis, and these subareas are schematically shown in Figure 3. The cumulative pipe lengths and service area of each analysis subarea are given in Tables 1 and 2. The average sewer density of the overall subareas is 164.9 feet/acre (123.5 m/ha).

For the purpose of the engineering analyses conducted in the prior infiltration/inflow study, the Neponset Valley Interceptor was subdivided into 81 segments. The interceptor segmentation along with the key manhole subareas and range of pipe diameter sizes are shown in Figure 4. The interceptor was subdivided such that the hydraulic properties along each segment including shape, size and slope, were reasonably constant.

4.3 Purpose/Issues

The Metropolitan District Commission Neponset Valley Interceptor is hydraulically overloaded during wet periods causing local back-up problems. Because of this situation, the MDC had initiated a preliminary feasibility study and construction plans for a new replacement sewer adja-

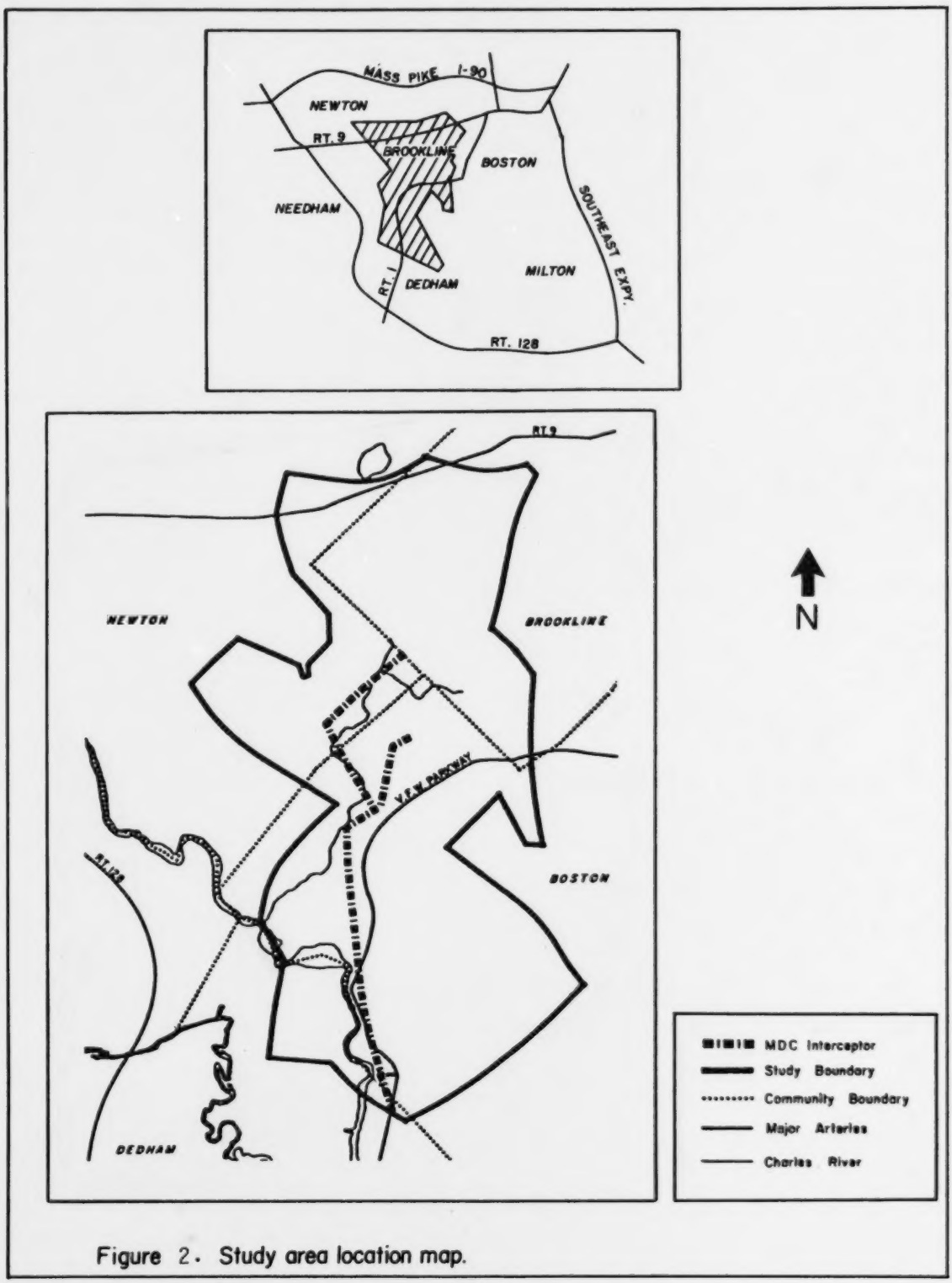


Figure 2. Study area location map.

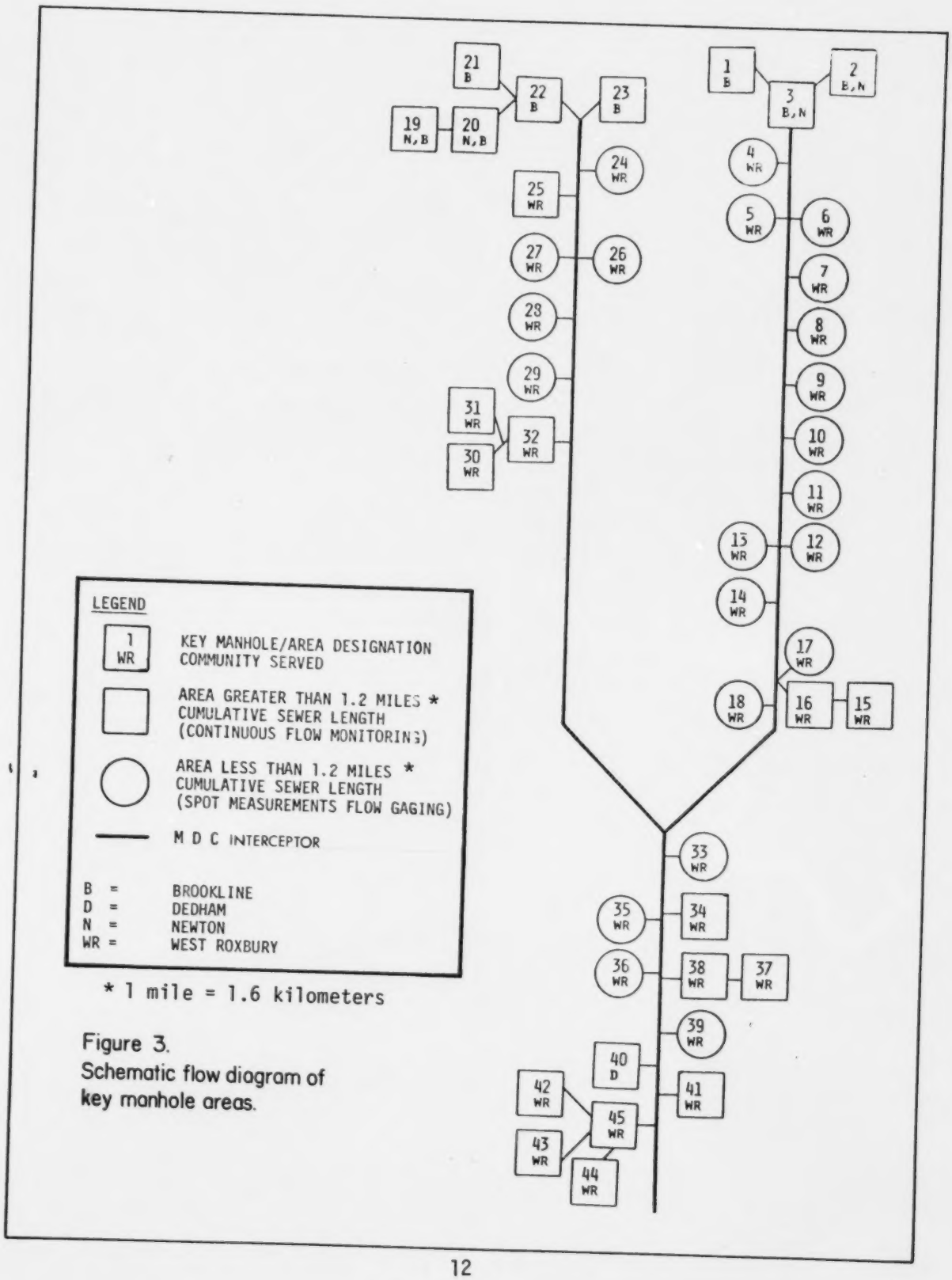


Figure 3.
Schematic flow diagram of
key manhole areas.

TABLE 1. PIPE LENGTH WITHIN EACH ANALYSIS AREA

AREA	Length of Sewer Pipe (Feet)	AREA NO.	Length of Sewer Pipe (Feet)
1	19,080	25	14,410
2	14,640	26	260
3	15,380	27	490
4	5,490	28	1,150
5	610	29	4,180
6	670	30	14,130
7	3,630	31	13,250
8	360	32	12,650
9	1,900	33	3,330
10	2,250	34	14,020
11	650	35	3,070
12	1,160	36	740*
13	2,160	37	21,060
14	1,410	38	16,430
15	15,610	39	1,370
16	15,500	40	16,980
17	990	41	17,130
18	1,310	42	7,730
19	8,800	43	19,060
20	24,010	44	5,610
21	16,220	45	2,580**
22	7,310		
23	14,260	Sub-Total	366,110 = 69.34 mi
24	3,280	MDC†	22,190
		Total	388,300 = 73.54 mi

Note:
1 m = 3.28 ft
1 mile = 1.6 kilometers

* abandoned sewers
** not monitored due to buried manholes
† Length of interceptor sections

TABLE 2. ACRES CONTAINED WITHIN EACH ANALYSIS AREA

AREA #	AREA (ACRES)	AREA NO.	AREA (ACRES)
1	96	23	110
2	38	24	58
3	96	25	100
4	38	26	6
5	5	27	8
6	7	28	4
7	19	29	26
8	5	30	83
9	9	31	90
10	13	32	90
11	3	33	19
12	8	34	82
13	6	35	19
14	13	36	6
15	84	37	120
16	70	38	140
17	70	39	6
18	6	40	96
19	200	41	100
20	130	42	45
21	150	43	120
22	51	44	32
		45	26
		TOTAL	2503*

Note:
1 hectare = 2.46 acre

* Total does not agree with total for study area, since large open spaces were not considered part of the individual analysis area.

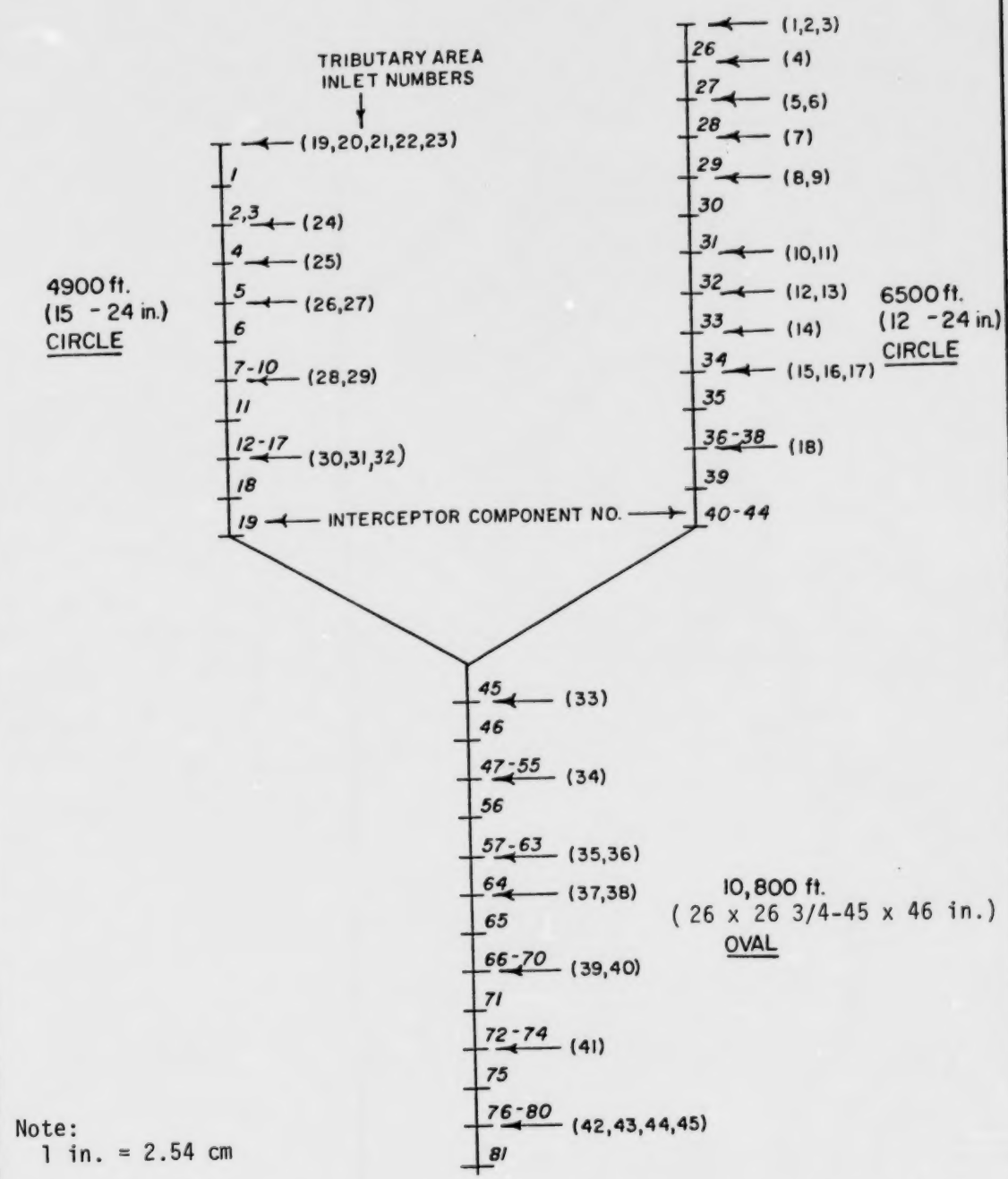


FIGURE 4. Schematic diagram of MDC Neponset Valley interceptor.

cent to the existing line. One goal of the prior study was to assess whether the existing system required complete replacement or would new relief segments at critical locations be sufficient. The infiltration/inflow analysis conducted on this area had been designed to assess three major conditions at sewer line connections to the MDC interceptor: 1) dry weather flow; 2) excess flow during high groundwater/infiltration periods; and 3) excess flows due to surface runoff or inflows.

4.4 Results of the Flow Gaging and Field Inspection Programs

An intensive field inspection and flow monitoring program was conducted on all key manholes plus numerous other sites during the period of March 23 to September 3, 1976. During that time, approximately 800 individual manhole inspections were conducted at 200 sites. A total of 45 key areas were flow monitored, utilizing both continuous liquid level type flow recorders (21 areas) and "spot" flow measurements for areas with little pipe length, under 1.2 mi (1.9 km) in length, with approximately one-half under 0.5 mi (0.8 km) in length.

A total of approximately 550 days of continuous flow data was generated along with 320 spot flow determinations. In conjunction with the flow gaging program an intensive calibration analysis was conducted to develop accurate and reliable stage rating curves for 31 of the key manholes. The remaining 14 key manholes were all spot measurement locations, including one site with less than 900 feet of sewer (insignificant discharge). Seven additional calibration curves were generated for additional upstream locations in some of the larger areas during various times in the study.

4.5 Field Calibration Program - Discharge Measurements

Stage/discharge quantities were field determined using direct methods for 31 of the key manholes, plus an additional 7 upstream sites. EDP performed this special field operation, since it believed that the straightforward application of Manning's equation using map slope information and literature values for the roughness coefficient, n, was too crude an approach for the estimation of dry weather flow, as well as for the estimation of infiltration/inflow conditions. A criteria was established that at least two and up to five good stage/discharge measurements were to be obtained for calibration of Manning's equation at each key manhole. This criteria was met except for extremely small collection systems where the dry weather flow would often be negligible. In those instances, only one good point, and sometimes none, were determined.

At 31 of the 45 key manholes direct stage/discharge determinations were made on background flow conditions taken at relatively steady state conditions during both dry weather and, in some cases, infiltration conditions. The field operation was designed such that the field crew would arrive at the same test segments at different times of the day, so as to maximize the chance of obtaining spread in the measured depths and discharges. For four key manholes, EDP performed a separate calibration operation using a

large specially designed water tanker* with a 2000 gal. (7.57 cubic m) capacity to artifically induce greater flow.

The flat-bed heavy duty truck was equipped with two 1000 gal. tanks that discharge through a four in. (10.6 cm) manifold installed with a four in. turbine water meter capable of direct reading flow rates up to 1000 gpm (63.1 l/s). The end discharge pipe is 3 in. (7.62 cm) in diameter, with a 3 in. quick action gate valve. The tanks were pressurized by a pneumatic air system so that constant pressure could be maintained in the range of 1-50 psi (35,155 kg/sq m). A constant discharge rate of up to 1.5 cfs (42.5 l/s) for sustained periods of time could be generated. The pressurized system was used to attain the higher flow rates. The tanker proved to be an valuable aid in generating a fairly wide range of controlled flow conditions for calibration purposes.

On arrival at a given test segment, the depth of flow and depth of sediment, if any, were carefully noted. Substantial sediment layers of sewage deposition were noted at some of the measurement sites, and proved to be a problem in the curve fitting process since they varied over the measurement program. Velocity determinations were made by measuring the time of travel of dye injected immediately upstream from the measurement site. Dye (Rodamine B or Uracine) was used in all cases. The length of the segment was measured in the field at all sites. At least two separate time of travel measurements were taken by noting the centroid of the injected dye wave for each flow condition, and, if necessary, more were taken until they converged narrowly to one value, which would then be used in computing the velocity at that particular depth.

The following procedure was used for the four sites where the water tanker was used for calibration purposes. Upon arrival at the site, the time of travel was recorded for the background flow. The tanks on the track were filled from nearby fire hydrants and pressurized while still connected to the hydrants. Water from the truck was discharged into the upstream manhole of the test segment, and the flow rate maintained for a sufficient period to allow stabilization of both the pressure in the tanks and the water depth in the downstream end of the pipe. The input flow depth in the sewer was recorded and the dye tests (time of travel) experiments were completed. Flows at lower depths were determined by progressively lowering the delivery rate from the truck. Using the input flow rate from the truck and an estimate of the background flow rate (dry weather flow depths were noted and velocity determined by dye tracer method), the total flow rate being routed through the pipe was determined within a small margin of error (±2%). These flow rates were used to check the velocity and flow determinations in the sewer.

All the field information with respect to the calibration effort is contained in Table 3. Column 1 gives the basin number, Column 2 gives the total number of depth versus flow points measured. In some cases one or

---

*The water tanker truck was developed as part of a U.S. EPA R&D sewer flushing study entitled, "Dry Weather Deposition and Flushing for Combined Sewer Overflow Pollution Control" (2).

TABLE 3. SUMMARY OF FIELD DISCHARGE CALIBRATION PROGRAM DATA

(1) Basin No.	(2) No. of Measur. Points	(3) No. of Good Points	(4) Source of Flow		(5) Pipe Size (in.)	(6) Measurements Levels (in.)	(7) Avg. Sediment Depth (in.)
			Back Ground	Truck			
1	2	2	X		12	4.0 & 4.25	None
2	2	2	X		12	4.0 & 7.75	3
3	4	3	X		12	3.7, 4.5 & 5.25	None
4	1	1	X		10	0.5	None
5	(*)						
6	(*)						
7	(*)						
8	(*)						
9	2	2	X		10	1.5 & 2.0	None
10	1	1	X		10	1.0	3
11	(*)						
12	1	1	X		10	1.0	1
13	(*)						
14	(*)						
15	6	5	X	X	12	3.75, 3.9, 4.75, 5, 6.5	4.5
16	2	2	X		12	4.0 & 4.5	1.5
17	(*)						
18	(*)						
19	3	3	X		15	4.75, 5.5 & 6.5	None
20	2	2	X		24	8.75 & 9.0	None
21	2	2	X		24	4.13 & 5.0	None
22	(*)						
23	3	3	X		15	3.25, 3.75 & 4.0	None
24	2	2	X		12	4.0 & 4.25	None
25	9	9	X	X	12	From 2.75 to 9.75	1.5
26	(*)						
27	1	1	X		8	0.5	None
28	2	2	X		8	1.5 & 2.25	None
29	2	2	X		8	1.5 & 1.75	None
30	8	5	X	X	10	5.25, 5.5, 6.75, 7.0 & 7.25	None
31	3	3	X		12	2.5, 3.25 & 3.5	None
32	5	4	X	X	12	5.5, 6.25, 6.75, 12.0	None
33	2	2	X		12	3.0 & 4.0	1
34	5	4	X		15	2.0, 2.5, 2.75 and 3.0	None
35	(*)						
36	1	1	X		12	2.5	2
37	3	3	X		15	4.25, 4.75 and 5.75	None
38	3	3	X		15	3.5, 4.0 and 4.5	None
39	(*)						
40	3	1	X		12	4.75	None
41	5	5	X		12	2.5, 3.125, 3.5, 3.75 & 6.0	2
42	3	3	X		15	4.0, 5.25 and 7.5	1.5
43	2	2	X		24	8.5 and 9.0	2
44	1	1	X		12	4.0	1
45	(*)						

(*) indicates basins without a calibrated Manning's equation. These basins do not have dipper measurements and often had negligible flow - no measurement.

Note: 1 in. = 2.54 cm

more of the measured points were clearly off as a result of a poor velocity determination. These points were eliminated and only those considered good were used in calibrating Manning's equation. The number of the good measurement points are shown in Column 3. Column 4 indicates the source of water, either background sewage flow or the background sewage flow plus clear water added from the water tanker. Column 5 gives the pipe size and Column 6 gives the depths at which the flows were measured. The last column indicates the average sediment depth over the number of observations. These sediment depths were accounted for in fitting Manning's equation by adjusting the depth/area of flow and hydraulic radius computations. In some basins the flow was negligible and therefore not measured. These cases are indicated by (*) in Column 2. In summary, 91 separate determinations of stage/discharge points were completed, of which 82 were termed acceptable or good points. Equipment breakdowns, erratic sediment beds and human error in travel time estimation account for the 9 clearly erroneous points.

SECTION 5

THEORECTICAL DETAILS FOR LEAST SQUARES APPROACHES

5.1 Foreword

Theoretical details of various least squares fitting procedures for using measured information to develop state/discharge curves are presented in Section 5.2. Various examples of stage/discharge relationships computed for differing assumptions from the prior infiltration/inflow study are compared with similar relationships derived by least squares methods in Section 5.3.

5.2 Theoretical Details

The use of Manning's equation to compute discharge in sewers from flow depth measurements is of universal practice in environmental engineering. In foot-second units, Manning's equation for computing discharge, Q, at a water depth, h, for a pipe of diameter, D, is given by:

$$Q = \frac{1.49\ r^{2/3}\ S^{1/2}a}{n} \qquad (1)$$

where:

Q = $f_1(h/D)$, flow rate (cfs),
r = $f_2(h/D)$, hydraulic radius (ft),
S = constant, energy slope (ft/ft),
a = $f_3(h/D)$, wetted area (ft^2), and
n = Manning's roughness coefficient.

Application of Manning's equation usually assumes steady uniform flow by setting the energy slope, equal to S_o, the pipe slope. It is also a common practice to assign a constant value for the roughness coefficient, n according to pipe material, age and state of maintenance. Values of 0.013 and 0.015 are often used in sewer system computations for concrete pipe.

One improvement to the straightforward application of Manning's equation is to adjust either the slope, S_o and/or the roughness coefficient n. An alternative procedure for establishing stage/discharge relationships is to utilize field stage/discharge calibration results in a least squares approach. More than one approach can be used to curve fit Manning's equation, (equation (1)), to observed data. Two of the more intuitive possibilities are: a) assume the slope is known and find the value of n that best fits the curve to the observed points; or b) assume n as known and determine slope $\hat{S}$ that best fits the curve to the observed points.

Extensive laboratory and field experiments (3, 4, 5 and 6) indicate that the value of n varies with the depth of flow in the pipe. A tabulation of the ratios, h/D, n/n_f and n associated with $n_f = 0.015$ is given in Table 4 (6). The value of n at full pipe (n_f) and its variations with depth was assumed known and the slope, $\hat{S}$ was taken as a surrogate for all the uncertainties with respect to $\hat{S}$ itself and the roughness coefficient, n. It was considered preferable to assume that n (or its variation with depth) is known and let a surrogate slope, $\hat{S}$ constant for all water depths, be the free parameter to be determined from the least squares procedure.

Other approaches can be used to fit Manning's equation to the data points. One intuitive approach is to optimize the roughness coefficient, n, instead of the slope, $\hat{S}$. The variability of n with h/D can be expressed by an appropriate mathematical expression and the optimization of the variable n function by non-linear means would be possible. The procedure to fit slope, $\hat{S}$, using least squares, is much simpler and was therefore adopted.

A number of the hydraulic parameters in Manning's equation can be expressed as a function of the ratio of observed flow depths, h_i, to pipe diameter, D, for measured values of discharge, Q_i.* Equation (1) is rewritten as:

$$Q_i = \frac{1.49\ r_i^{2/3}\ S^{1/2}\ a_i}{n_i} \qquad (2)$$

where:

$Q_i = f(h_i/D)$, measured flow rate (cfs),

$r_i = f(h_i/D)$, hydraulic radius (ft),

S = constant, energy slope (ft/ft),

$a_i = f(h_i/D)$ wetted area (ft^2),

$n_i = f(h_i/D)$, Manning's roughness coefficient,

h_i = observed water depth in the pipe, (ft), and

D = pipe diameter, (ft).

A number of least squares approaches are possible depending on how the minimization problem of fitting the observed values of (h_i, Q_i) about Manning's equation is posed. Three possible approaches are presented here.

The first approach is the standard ordinary least squares formulation where the standard deviation of the residuals (observed-predicted) is independent of the magnitude of discharge. The resulting expression for the sum of the squares of the residuals, $O(S)$, can be written as:

---

* See Section 5.4 for sediment considerations.

TABLE 4. TABULATION OF MANNING'S ROUGHNESS COEFFICIENT VERSUS DEPTH OF FLOW

h/D	n/n_f	n*
0.050	1.190	0.01785
0.075	1.215	0.01823
0.100	1.240	0.01860
0.125	1.250	0.01875
0.150	1.260	0.01890
0.175	1.267	0.01901
0.200	1.275	0.01913
0.225	1.278	0.01917
0.250	1.280	0.01920
0.275	1.285	0.01928
0.300	1.290	0.01935
0.325	1.285	0.01928
0.350	1.280	0.01920
0.375	1.275	0.01913
0.400	1.270	0.01905
0.425	1.265	0.01898
0.450	1.260	0.01890
0.475	1.251	0.01877
0.500	1.243	0.01865
0.525	1.237	0.01856
0.550	1.230	0.01845
0.575	1.225	0.01837
0.600	1.220	0.01830
0.625	1.210	0.01815
0.650	1.200	0.01800
0.675	1.190	0.01785
0.700	1.180	0.01770
0.725	1.170	0.01755
0.750	1.160	0.01740
0.775	1.147	0.01721
0.800	1.135	0.01703
0.825	1.111	0.01676
0.850	1.100	0.01650
0.875	1.084	0.01626
0.900	1.068	0.01602
0.925	1.056	0.01584
0.950	1.045	0.01568
0.975	1.022	0.01533
1.000	1.000	0.01500

* Based on $n_f = 0.015$

$$\underset{S}{\text{Min}}\ O(S) = \sum_{i=1}^{m} [Q_i - S^{1/2} K_i (h_i/D)]^2 \qquad (3)$$

where: Q_i = observed values of flow from the calibration and

$$K_i(h_i/D) = \frac{1.49\ a_i\ r_i^{2/3}}{n_i}\ ; \qquad (4)$$

m = number of observed Q_i; and all other variables as defined before.

To minimize O(S) its derivative is taken with respect to S and set equal to zero. The resulting expression for the "best-fit" $\hat{S}$ is:

$$\hat{S}^{1/2} = \frac{\sum_{i=1}^{m} K_i\ Q_i}{\sum_{i=1}^{m} K_i^2}\ . \qquad (5)$$

In the second approach, the variance of the measured value, Q_i, from its computed value is assumed to be proportional to the magnitude of Q. The objective function to be minimized under this assumption is as follows:

$$\underset{S}{\text{Min}}\ O(S) = \frac{1}{S} \sum_{i=1}^{m} \frac{1}{K_i} [Q_i - K_i\ S^{1/2}]^2 \qquad (6)$$

The resulting estimate of slope, $\hat{S}$, is given by:

$$\hat{S}^{1/2} = \frac{\sum_{i=1}^{m} Q_i}{\sum_{i=1}^{m} K_i}\ . \qquad (7)$$

Another feasible model would be to assume that the standard deviation of Q_i is proportional to Q_i. This assumption is the same as a constant coefficient of variation. The resulting objective function, O(S) is given by:

$$\underset{S}{\text{Min}}\ O(S) = 1/S^2 \sum_{i=1}^{m} 1/K_i^2\ (Q_i - K_i S^{1/2})^2\ . \qquad (8)$$

The resulting estimate of $\hat{S}$, is given by:

$$\hat{S}^{1/2} = \frac{1}{m} \sum_{i=1}^{m} Q_i/K_i\ . \qquad (9)$$

The choice of a particular model to use in any given application is somewhat arbitrary. The ordinary least squares approach given by equation (5) weights large observations of (Q_i, K_i) too heavily, while the estimate of $\hat{S}$, resulting from the third approach given by equation (9) weights large observations too little. The second approach which assumes that the variance of the observed Q about its true value is proportional to its magnitude, seems to provide a reasonable compromise.

Pertinent information illustrating the least squares fitting procedures are presented in Table 5. Depth of flow and velocity measurements in a 27 in. vcp sanitary trunk sewer in the Boston area were determined on four separate occasions. Depth of flow and measured velocities using a Marsh-McBirney meter are given under column A in Table 5. Estimates of the hydraulic radius, the wetted cross-sectional area, and Manning's roughness coefficients associated with each measured depth of flow are given under column B in Table 5. The tabulation of variable Manning's roughness coefficient for n_f = 0.015 given in Table 4 was used to prepare the estimates shown in Table 5. Estimates of discharges Q_i, K_i and K_iQ_i are given under column C in Table 5.

The ordinary least squares estimate of the surrogate slope parameter $\hat{S}$, using equation (5) is computed as follows:

$$\hat{S} = \left[\frac{562.557}{9902.724}\right]^2 = 0.00323$$

The weighted least squares estimate of the surrogate slope parameter using equation (7) is computed as follows:

$$\hat{S} = \left[\frac{8.746}{161.925}\right]^2 = 0.00292$$

Results of a sensitivity analysis using the two alternative least squares approaches on different sets of data derived by various representative measurement error assumptions are presented in Table 6. Four different cases of measurement error for the field data shown in Table 5 are considered. The first case entails a positive velocity meter bias of 10% for all four observations. An erroneous positive depth of flow error of one in. for all observations is considered in the second case. An erroneous positive depth of flow error of one in. for only the high flow measurements is depicted for the third case. The last case considers both depth of flow error and positive velocity meter bias for the high flow measurements. Many other combinations of measurement error are also possible. The fitted slopes using the simple ordinary least squares (ols) and the weighted least squares (wls) approaches on the four different data sets are given under columns A and C in Table 6. The square root of the ratio of the fitted ols and wls slopes for each case to the nominal estimates (no error) are given under columns B and D in Table 6. These ratios reflect the increase of hydraulic conveyance about the nominal estimate assuming that the derived relationships are thereafter used without error, i.e., correct depths of flow are entered into the computations. The results show that for these data sets either method will

TABLE 5. SAMPLE CALCULATIONS OF LEAST SQUARES PROCEDURES

A. Measured Information		B. Physical Data			C. Calculations			
Depth of Flow* (inches)	Measured Velocity (fps)	r_i (ft)	a_i (ft^2)	n_i**	Q_i (cfs)	K_i***	K_i^2	K_iQ_i
9.5	2.50	.437	1.249	.0192	3.123	55.868	3121.233	174.464
11.5	2.90	.504	1.614	.0190	4.680	80.282	6445.200	375.743
4.75	1.15	.242	.471	.0190	0.542	14.322	205.120	7.756
4.25	1.10	.219	.401	.0189	0.401	11.453	131.171	4.594
Sum					8.746	161.925	9902.724	562.557

(Cubic meters/second = 0.0283 cfs)

Notes:

* 27 in. vcp trunk sewer with no sediment, Plan & Profile slope = 0.0036
** variable Manning's roughness coefficient with $n_f = 0.015$
*** $K_i = \frac{1.49\ a_i\ r_i^{2/3}}{n_i}$

TABLE 6. SENSITIVITY ANALYSIS OF LEAST SQUARES PROCEDURES

Case	Ordinary Least Squares-Eq.(5) A.	B.	Weighted Least Squares-Eq.(7) C.	D.
- Existing Data (Nominal Estimate)	.00323	1.00	.00292	1.00
1. Velocity Measurement Increase (10%) for all Observations	.00391	1.10	.00353	1.10
2. Depth of Flow Increase (1 inch) for all Observations	.00286	0.94	.00253	0.93
3. Depth of Flow Increase (1 inch) for 1st and 2nd Observations	.00294	0.95	.00270	0.96
4. Depth of Flow Increase (1 inch) and Velocity Increase (10%) for 1st and 2nd Observations	.00354	1.05	.00321	1.05

A. Ordinary least squares estimate of slope.

B. Square root (estimate slope for stated data/estimated nominal slope). Slopes computed by ordinary least squares formulation - equation (5).

C. Weighted least squares estimate of slope.

D. Square root (estimated slope for stated data/estimated nominal slope). Slopes computed by weighted least squares formulation - equation (7).

yield comparable rate of errors. The greatest degree of error derives from the first case involving velocity measurement bias.

Estimates provided by the weighted least squares approach are nevertheless preferred, particularly when very high (full pipe) flow conditions are gaged with the possibility of both velocity and depth of flow measurement errors. The weighted least squares approach tends to lessen the impact of large (Q_i, K_i) values. A good compromise in practice would be to calculate the slopes by both methods and then compute the geometric average of the two estimates.

A comparison of estimated discharges in the 27 in. trunk sewer computed using plan and profile slopes of 0.003 with fixed roughness coefficients of 0.013 and 0.015 with estimates computed using both of the least squares approaches for various flow depths is presented in Table 7. The calculations show that discharges computed using plan & profile slope with fixed roughness coefficients (cases A and B) greatly exceed those computed by either of the least squares procedures. It turns out for this example that the differences shown in Table 7 between cases (A and B) and (C and D) are considerably decreased if the variable roughness coefficient formulation is used for cases (A and B).

5.3 Comparison of Stage Rating Curves From Prior Study

Various stage/rating curves for five key manholes in areas 2, 25, 28, 32 and 41 in the study area are presented in Figures 5 through 9. Field discharge measurements are also shown. For each of the five sites, four alternative curves are plotted for the following conditions: 1) minimum square fit (wls) for $\hat{S}$ assuming variable n, (n_f = 0.015); 2) minimum square fit (wls) for $\hat{S}$ assuming a constant n = 0.015 and considering sediment, if present; 3) clean pipe, with slope from the plans and a constant n = 0.013; and 4) clean pipe with slope from the plans and a constant n = 0.015.

It is common practice in flow measurement studies to use directly the pipe slope from as-built drawings in Manning's equation and to disregard the presence of sediments in the estimation process. It is implicitly assumed that the pipe slope is known with certainty and that the hydraulic flow profile slope equals the pipe slope, i.e., the water slope is parallel to the pipe slope. These important assumptions underlie the computational process used to derive curves 3 and 4. The least squares fitting process used to establish curves 1 and 2 makes no such limiting assumptions but instead uses actual flow measurements impacted by sediment beds and both pipe and water slope irregularities to develop realistic rating curves.

If the least-square curve with variable n (curve 1) is considered to closely represent the true flows in the pipe, then one can estimate the joint error incurred in assuming the pipe slope from drawings and constant values of n(curves 3 and 4). For n = 0.013, for example, these "errors" are represented by ratios presented in Table 8. All ratios shown in Table 8 would be reduced by about 14% if a fixed roughness coefficient of n = 0.015 were used.

TABLE 7
COMPARISON OF DISCHARGES COMPUTED USING
VARIOUS STAGE/DISCHARGE FORMULATIONS

Assumed Depth of Flow (in.)	Discharges (cfs)				Ratio of		
	A	B	C	D	A/D	B/D	C/D
10	4.98	4.22	3.50	3.33	1.495	1.267	1.051
18	13.10	11.43	9.98	9.55	1.372	1.197	1.045
25	18.27	15.86	15.64	14.80	1.234	1.072	1.057

27in. Trunk Sewer (Cubic meters/second = 0.0283 cfs)

KEY:

A - Manning's Equation, Plan & Profile Slope = 0.003, Fixed n = 0.013.

B - Manning's Equation, Plan & Profile Slope = 0.003, Fixed n = 0.015.

C - Manning's Equation, Fitted Slope (ols), Variable n with n_f = 0.015.

D - Manning's Equation, Fitted Slope (wls), Variable n with n_f = 0.015.

Note: 1 in. = 2.54 cm

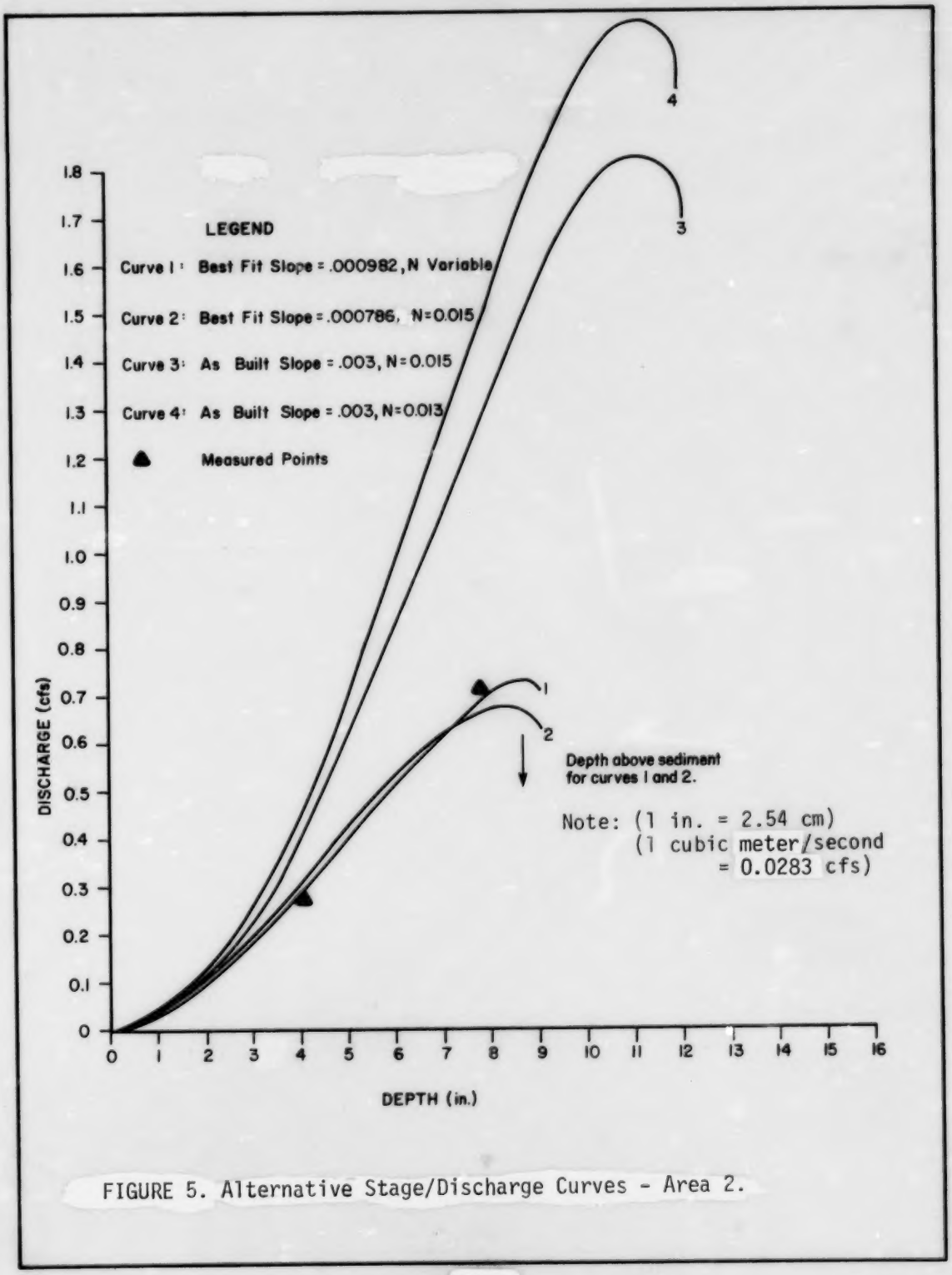


FIGURE 5. Alternative Stage/Discharge Curves - Area 2.

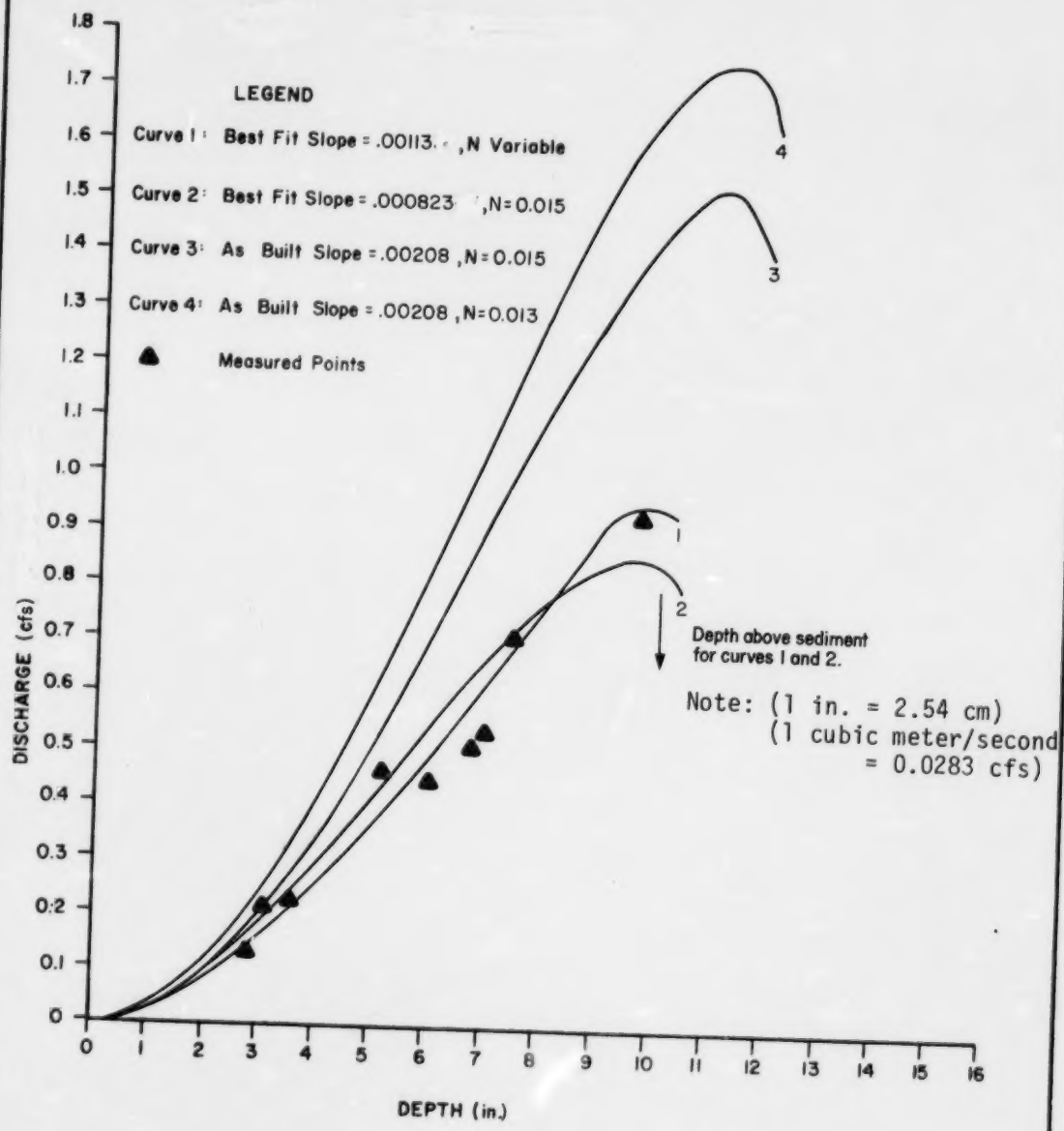


Figure 6. Alternative Stage/Discharge Curves - Area 25

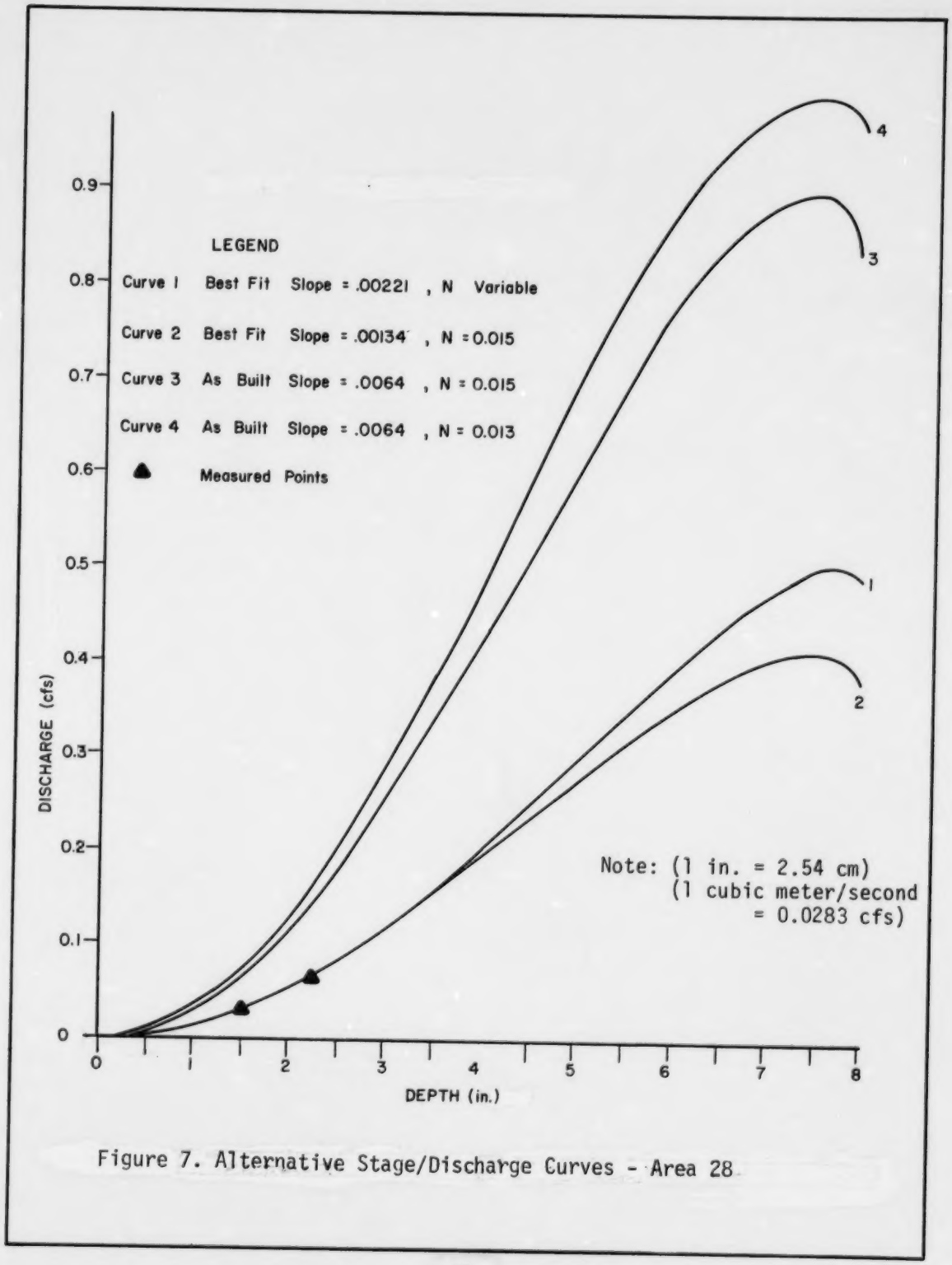


Figure 7. Alternative Stage/Discharge Curves - Area 28

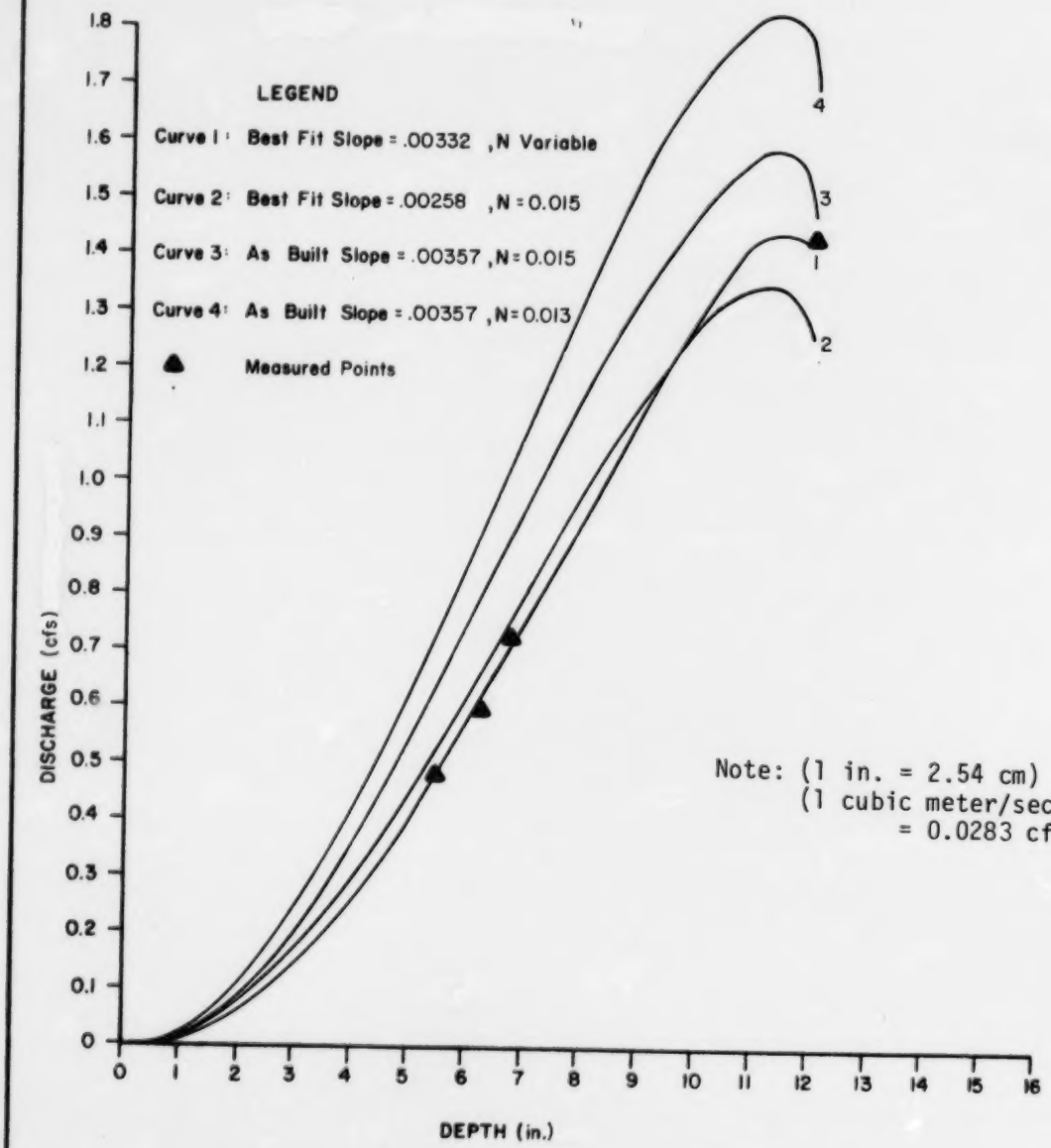

LEGEND
Curve 1: Best Fit Slope = .00332 , N Variable
Curve 2: Best Fit Slope = .00258 , N = 0.015
Curve 3: As Built Slope = .00357 , N = 0.015
Curve 4: As Built Slope = .00357 , N = 0.013
Measured Points
DISCHARGE (cfs)
DEPTH (in.)
Note: (1 in. = 2.54 cm)
(1 cubic meter/second = 0.0283 cfs)

Figure 8. Alternative Stage/Discharge Curves - Area 32

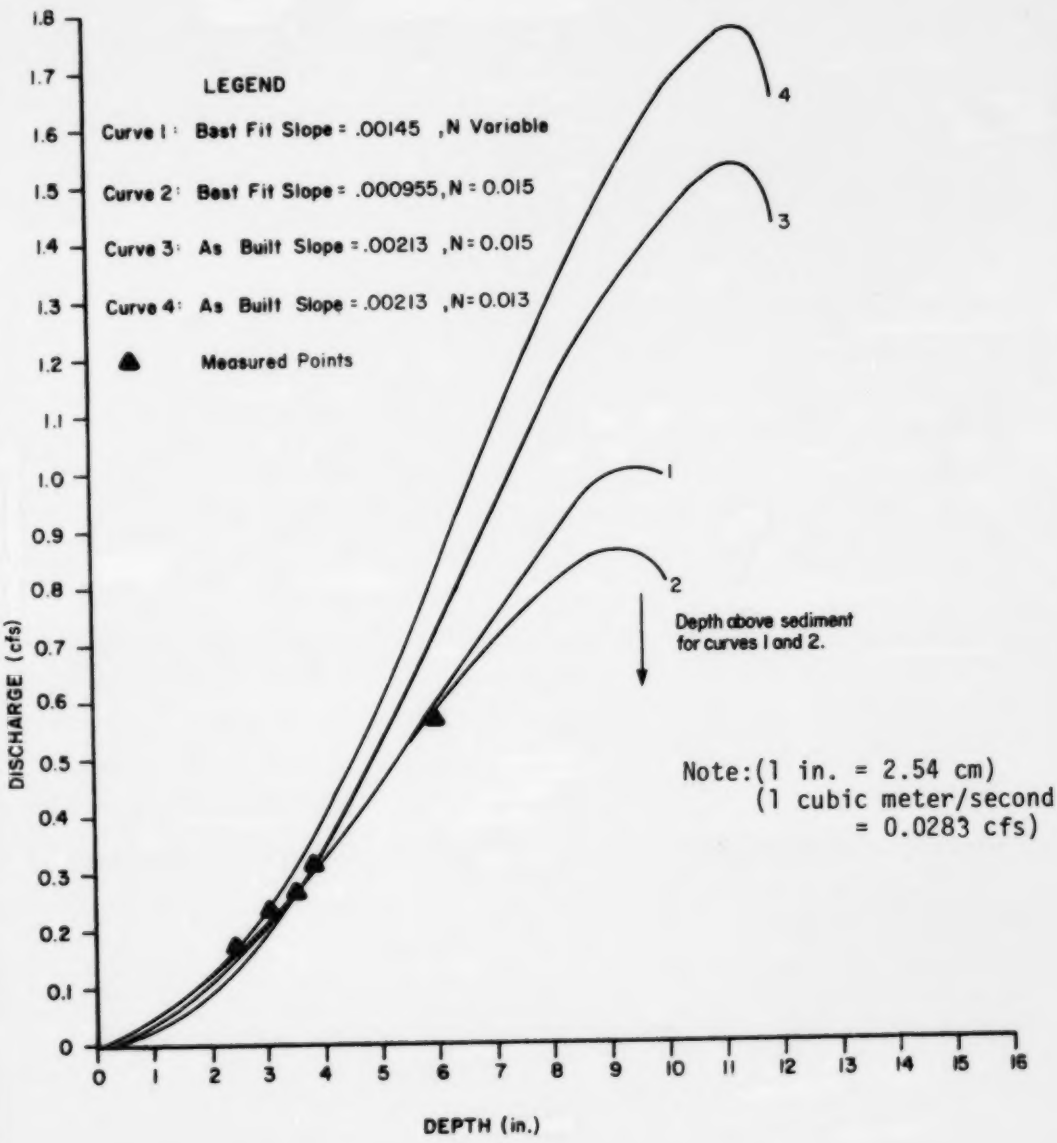


Figure 9. Alternative Stage/Discharge Curves - Area 41

Table 8

RATIOS OF FLOW COMPUTED USING PIPE SLOPE FROM DRAWINGS WITH FLOWS COMPUTED FROM LEAST SQUARE FIT OF CALIBRATION DATA

Depth (in.)	AREA NUMBER 2*	25*	41*	32	28
1	1.56	1.22	0.78	1.43	2.40
2	1.36	1.25	0.96	1.66	2.53
3	1.48	1.40	1.11	1.69	2.46
4	1.58	1.53	1.21	1.68	2.44
5	1.75	1.62	1.32	1.56	2.37
6	1.91	1.67	1.44	1.51	2.24
7	2.03	1.68	1.48	1.46	2.07
8	2.18	1.69	1.50	1.42	1.95
9	2.55	1.70	1.66	1.37	-
10	-	1.73	-	1.32	-
11	-	-	-	1.28	-
12	-	-	-	1.20	-

* Indicates the presence of sediment in the pipe.

Note: This table compares discharges computed by plan & profile slope with constant roughness coefficient (n = 0.013) with discharges estimated using (wls) slope and variable roughness coefficient (n_f=0.015).

The results for these five areas show the pipes have less transport capacity than one would normally expect.* All measured points fall below curve 4, for n=0.013, and almost always below curve 3 for n= 0.015. In practice, this can be explained by discrepancies between the true field elevations and those shown in drawings, poor alignment of pipes, the presence of sediments and the assumption of fixed roughness coefficient.

Two other factors that may have a considerable bearing on estimating flow in a sewer pipe segment are the pipe slope differences over a long run of sewer in which the measurement segment is contained and backwater effects. In the first case, the actual hydraulic water profile slope in a given run of sewer pipe may not resemble the measurement segments's pipe slope even if known with perfect certainty. This type of boundary condition problem can be partially mitigated by carefully selecting the measurement segment, and/or if not possible, by iterative selections of "trial-error" overall sewer run slope, until estimated flows agree with predicted flows. This problem is implicitly dealt with in the least squares fitting process. An additional boundary condition factor which may partially account for some of the differences shown in Table 8 is the presence of backwater conditions.

* Comparison of stage/discharge curves from other key manholes within the study area showed the reverse to be true. These results are given in Section 6.

Manning's equation is based upon free-flowing open channel conditions and must be corrected for backwater effects. This correction is difficult to accomplish in practice due to measurement complexity and the ensuing backwater computations. Excessive backwater flow conditions were noted in some portions of the sewerage system, particularly in the MDC interceptor during wet periods. Slight backwater effects may have been present at several key manholes during dry weather when the flow calibration program was conducted. No backwater conditions were observed for most of the key manholes gaged during this period. All of these factors, taken individually and/or collectively, can affect both the friction factor n, the pipe slope S_o, and ultimately the hydraulic flow profile slope.

It is extremely difficult to ascertain with any certainty how much of the difference is caused by the pipe/water slope and how much by the friction factor n, even disregarding the imperfections of Manning's equation itself. In order to show that the discrepancies of flows evidenced at these five sites are not unreasonable, assume that those discrepancies are caused solely by error in the pipe slopes. Using the pipe slope and segment length, the absolute error in pipe elevation between the upstream and downstream manhole causing the average difference between curves 1 and 3 shown in Figures 5-9 was computed by Manning's equation. These results are shown in Table 9. Curve 1 was computed using least squares method with variable roughness factor (n_f = 0.015) and curve 3 was computed using plan and profile slope with fixed roughness coefficient (n = 0.015). Visual inspection of stage/discharge curves for methods 2 and 3 in Figures 5-9 indicate comparable differences as those shown in Table 9.

Table 9.

ESTIMATION OF ELEVATION ERROR CAUSING AVERAGE ERROR BETWEEN CURVES 1 & 3 (FIGURES 5-9) FOR AREAS 2, 25, 28,32 and 41

Areas	Slope from Drawings	Segment Length (Ft)	Error in Difference of Elevation (In.)
2	0.00300	200.0	4.7
25	0.00208	259.3	3.5
28	0.00640	125.0	0.7
32	0.00357	182.0	3.8
41	0.00213	141.0	1.3

Note: 1m = 3.28 ft 1 in.= 2.54 cm

Errors of magnitude shown in Table 9 can easily result from construction and settlement of manholes, and in some cases even by the presence of sediment. The likelihood of erroneous flow estimation increases with short and flat pipes. Of all the key manhole locations for which a calibration curve was computed in the prior study, the fitted slope was greater, while

in 7% it was within 10% of the values from the drawings. It should be noted that a fitted slope higher than the drawing slope does not necessarily mean higher flows than those given by curves 3 or 4. This is because in the curve fitting process the value of n at full pipe was taken equal to 0.015, varying up to a maximum value of 0.01935 at h/D = 0.3. This difference would, in most cases off-set the increased slope.

Comparison between curves 1 and 2 in Figures 5 through 9 show that although both curves comply with mathematical conditions that the sum of squares of the deviations from the observed values is minimum, curve 1 seems to adhere better to the observed points. This fact can be interpreted as additional evidence that Manning's equation is more capable of reproducing the physical phenomena of flow in pipes, if n is considered as a function of depth of flow.

5.4 Sediment Considerations

The presence of sediment beds can be considered in the least squares curve fitting procedure by appropriate adjustment of the hydraulic parameters defined in equation (4). The cross sectional area, a_j, and the hydraulic radius, r_j, are directly computed using the top of the sediment bed as the reference level. The variable roughness factor tabulation given in Table 4 is based on a circular cross section. Estimates of the variable roughness factor can be generated for non-circular cross section. Estimates of the variable roughness factor can be generated for non-circular cross sections (circular/non-circular with/without sediment) using equivalent circular estimates. First, an equivalent diameter is computed for the total cross section free from sediments. Next, an equivalent depth of flow (circular section) is computed using the wetted area of the non-circular section for each flow observation. Lastly, the appropriate estimate of variable n is found in Table 4 using the aforementioned equivalent circular depth of flow to equivalent diameter estimates.

SECTION 6

DISCHARGE COMPUTATIONS

6.1 Foreword

Details of three alternative stage/discharge procedures for estimating peak dry weather flow, infiltration and inflow conditions for the 45 key manhole gaging locations in the study area are presented in Section 6.2. Flow estimates at the 45 key manholes are presented in Section 6.3 using each of the three alternative procedures.

6.2 Alternative Stage/Discharge Formulations

For the purposes of this study, three different sets of stage/discharge curves were generated for each of the 45 key manhole gaging locations. The three "what if" conditions assumed in developing the stage/discharge curves are as follows:

- method 1: Manning's equation with slope computed by least squares (wls) and variable n (n_f = 0.015);
- method 2: Manning's equation with plan & profile slope and variable n (n_f = 0.015); and
- method 3: Manning's equation with plan & profile slope and fixed n = 0.015.

These three conditions considered in generating discharge from stage information collected from the field program are realistic. The aim of the analysis in this study is to compare the dwf, infiltration and inflow discharge estimates and the associated rehabilitation program costs using method 1 with those computed using methods 2 and 3. The difference in rehabilitation costs resulting from using method 1 versus methods 2 and 3 will then be weighted against the additional cost of obtaining flow discharge calibration points and performing the least square analysis.

Stage/discharge curves for the three methods were generated for each of the 45 key manhole locations. A representative display of these curves are shown in Figures 10 through 14 for areas 1, 4, 20, 23, and 43, respectively. Note that curve 1 (least squares method) lies above curves 2 and 3 (plan & profile slope) for area 20 and below for areas 1, 4, 23 and 43. Note also that curve 1 is significantly different (with the exception of area 1, Figure 10) than curves 2 and 3 in Figures

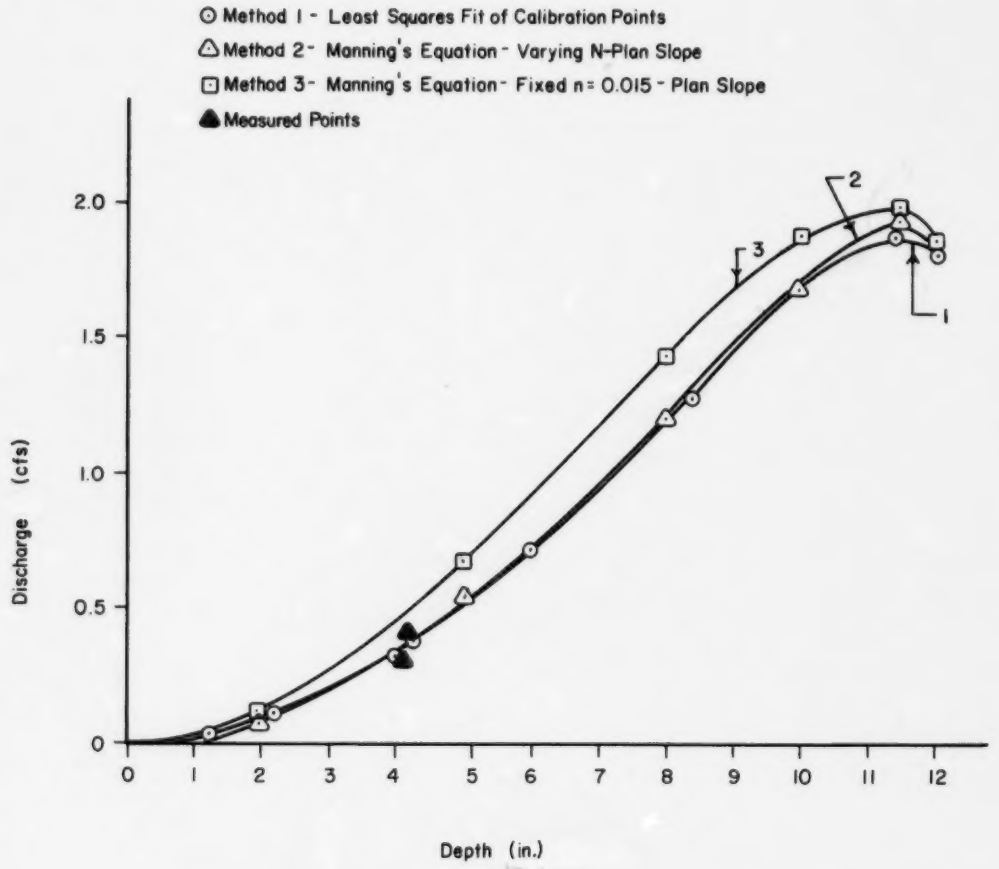


Figure 10. Stage/discharge curves for area 1.

Note: (1 in. = 2.54 cm)
(1 cubic meter/second = 0.0283 cfs)

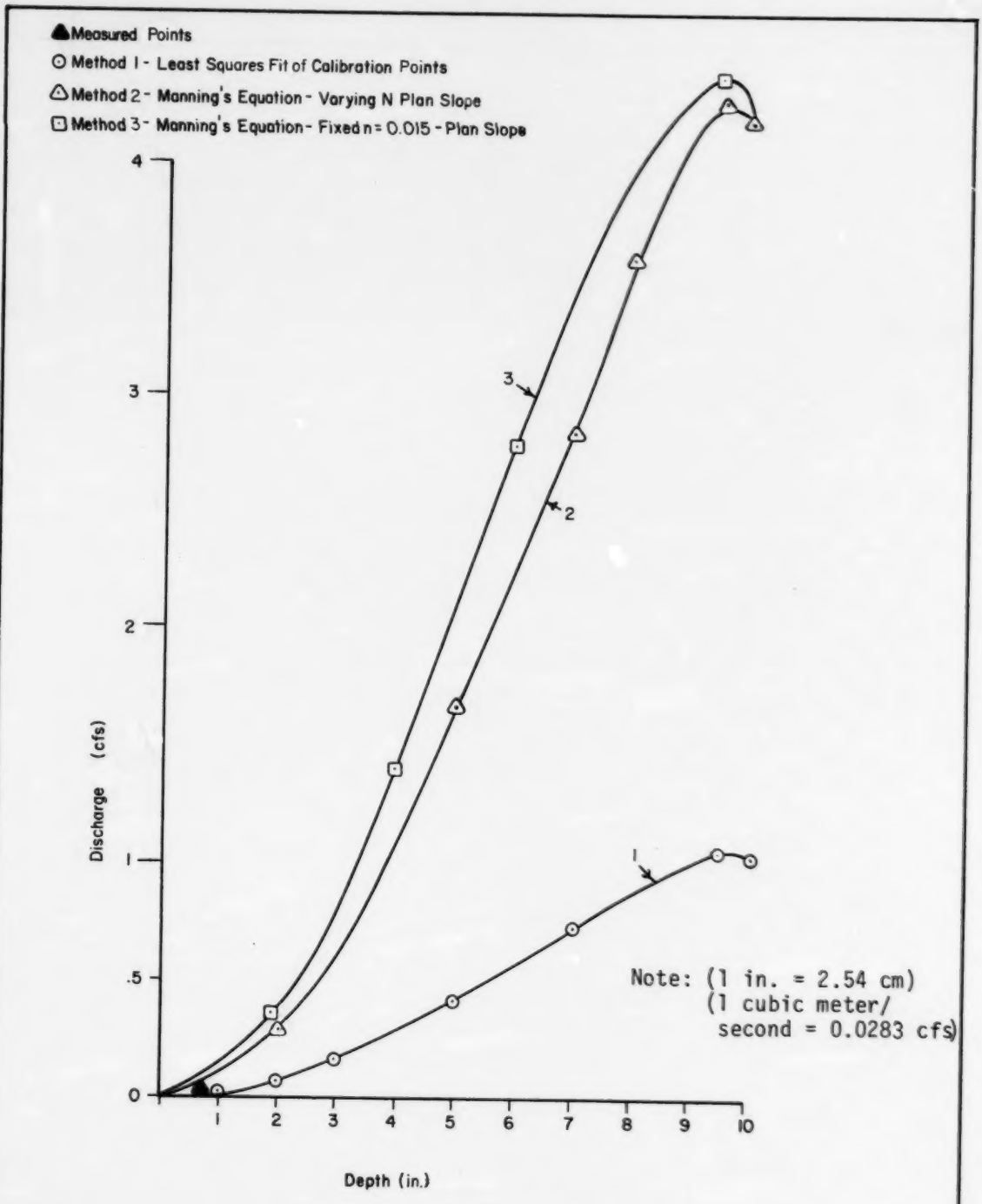


Figure 11. Stage/discharge curves for area 4.

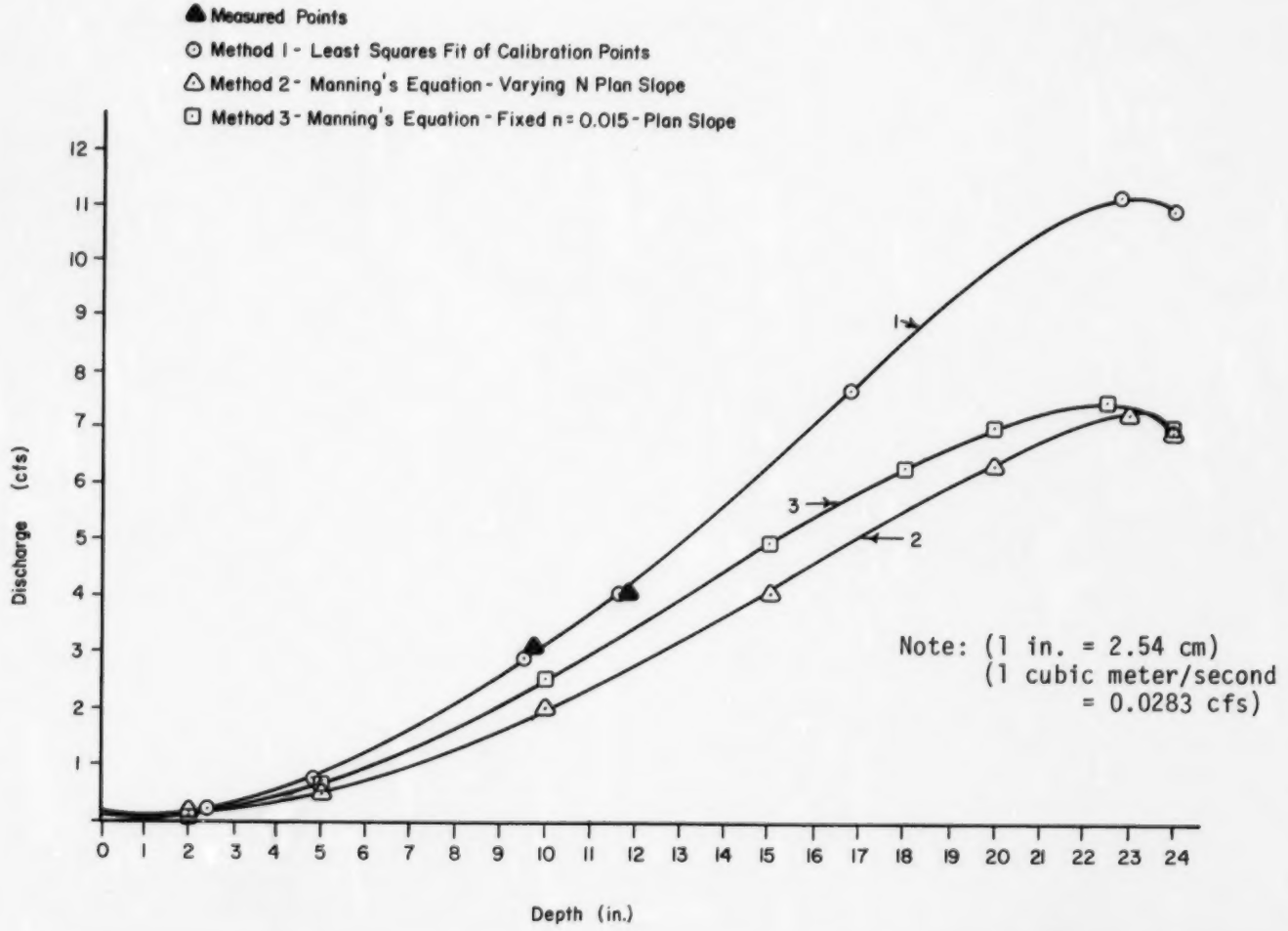

Measured Points
Method 1 - Least Squares Fit of Calibration Points
Method 2 - Manning's Equation - Varying N Plan Slope
Method 3 - Manning's Equation - Fixed n = 0.015 - Plan Slope
Discharge (cfs)
Depth (in.)
1
2
3
Note: (1 in. = 2.54 cm)
(1 cubic meter/second
= 0.0283 cfs)

Figure 12. Stage/discharge curves for area 20.

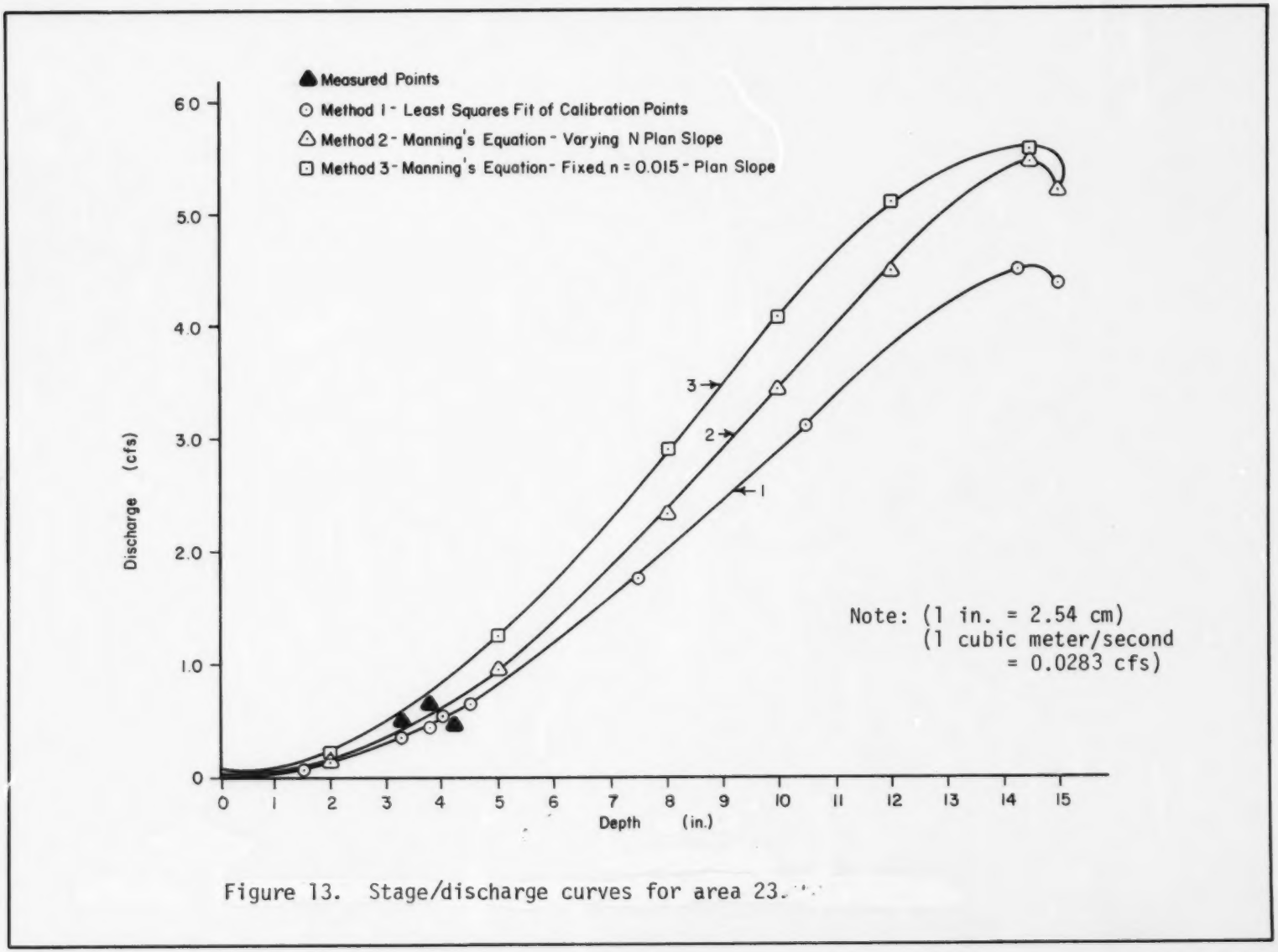


Figure 13. Stage/discharge curves for area 23.

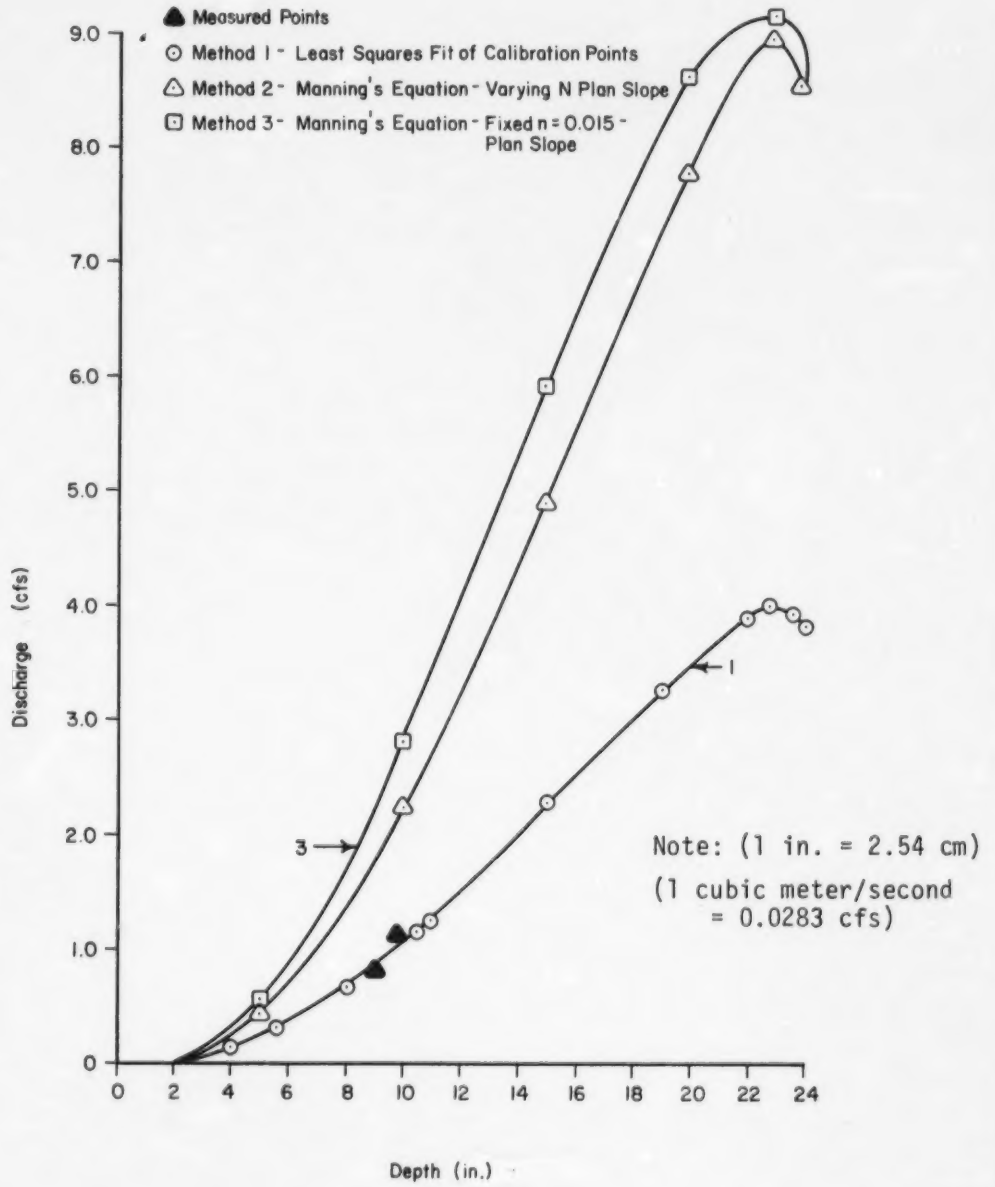


Figure 14. Stage/discharge curves for area 43.

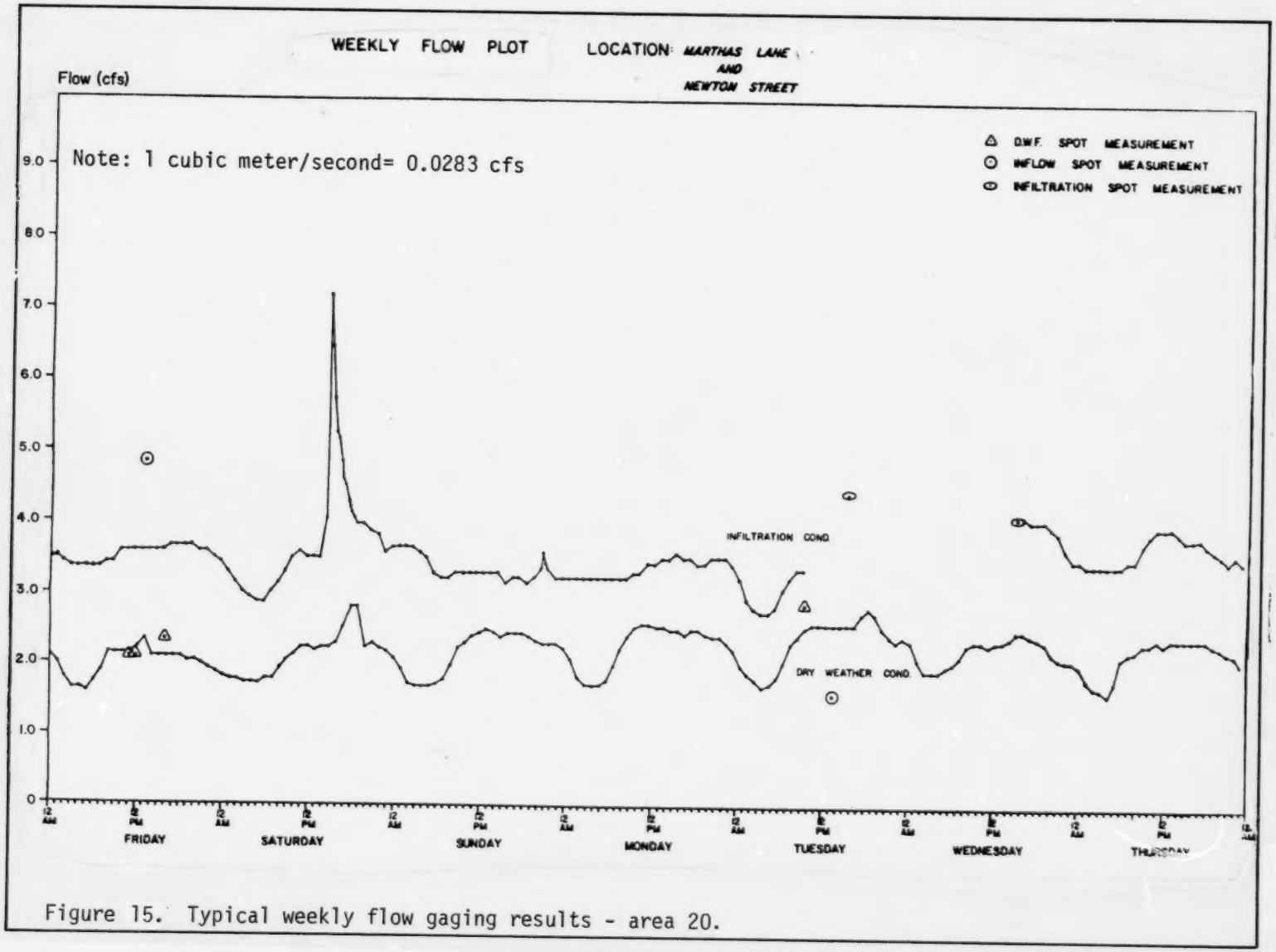


Figure 15. Typical weekly flow gaging results - area 20.

11 through 14. The differences between curves 2 and 3 are also appreciable for all locations.

The three methods were used to translate the field engineering program data collected during the prior study into discharge information. Discharge results for method 1 were available from the prior study (1), but were slightly modified after a thorough analysis of the field stage and the stage/discharge relationships. For the other two methods, a total of 550 days of continuous sewage levels generated at 21 stations with automatic recorders and 320 spot determinations were transcribed. A typical weekly plot of discharge information is presented in Figure 15 showing continuously recorded dwf and infiltration conditions along with several spot measurements. Discharge levels shown in Figure 15 were computed using Method 1.

6.3 Discharge Estimates

Peak dwf, peak excess infiltration and peak excess inflow for the 21 key manholes where continuous recorders were installed were computed in the following manner. First, the daily volumes of dwf were determined to establish day-of-week typical volumes. The maximum daily volume was converted to rate to establish peak dwf. Secondly, daily volumes during infiltration periods were compared with corresponding day-of-week dry weather daily volumes to determine peak daily excess infiltration volumes. Maximal excess infiltration daily volumes were then computed to rate to establish excess infiltration rates. Thirdly, peak instantaneous inflow rates were compared with comparable time-of-day/day-of-week dwf rates to establish peak excess inflow rates.

Spot measurements at the other 24 key manholes for the various conditions of dwf, infiltration and inflow were adjusted to comparable time-of-day/day-of-week using relationships derived through analysis of the continuous records. Peak dwf, excess infiltration and peak excess inflow were then computed.

The flow analysis using the three different stage/discharge methods are summarized in Tables 10, 11 and 12, respectively. The information presented in these tables are as follows: a) interceptor component number (see Figure 4, page 10); b) tributary area designations (see Figure 4); c) present MDC interceptor hydraulic capacity (cfs) full pipe conditions-no surcharge; d) measured peak dwf (cfs); e) measured excess infiltration (cfs); and f) measured excess inflow (cfs). All measured flows presented by area have been cumulated so that direct visual comparison to the MDC interceptor capacity, segment by segment, can be made. Computation of the interceptor hydraulic capacity was accomplished through use of a computer simulation model, calibrated against known key manhole measurements.

With reference to Figures 2 and 4, the interceptor can be visualized as a "Y" system. Interceptor segments 1-19 form the upper westerly portion, while segments 20-44 form the upper easterly portion. The two portions converge at segment 45 and the interceptor continues through segment 81.

TABLE 10.

SUMMARY OF CONTRIBUTORY DISCHARGES TO MDC INTERCEPTOR: METHOD 1

Interceptor Component	Direct Tributary Areas	Present Interceptor Capacity (cfs)	Peak DWF (cfs)	Excess Infiltration (cfs)	Excess Inflow (cfs)
1	19-23	4.57	3.75	2.10	5.80
2-3	-	4.57	3.75	2.10	5.80
4	24	4.26	3.96	2.15	6.17
5	25	5.01	4.18	2.34	6.88
6	26,27	7.96	4.18	2.34	6.98
7-10	-	4.47	4.18	2.34	6.98
11	28,29	4.49	4.25	2.71	7.06
12-17	-	7.47	4.25	2.71	7.06
18	30-32	9.64	4.77	3.81	8.29
*19	-	9.61	4.77	3.81	8.29
20-26	1-3	6.49	1.88	1.18	5.52
27	4	3.81	1.98	1.28	5.57
28	5,6	5.06	2.00	1.30	5.65
29	7	5.03	2.01	1.31	5.74
30	8,9	2.93	2.05	1.34	6.28
31	-	2.53	2.05	1.34	6.28
32	10,11	2.53	2.06	1.35	6.86
33	12,13	2.51	2.10	1.37	7.33
34	14	2.51	2.20	1.39	7.33
35	15-17	5.13	2.93	1.65	8.32
36-38	-	4.19	2.93	1.65	8.32
39	18	5.11	2.94	1.66	8.41
**40-44	-	5.12	2.94	1.66	8.41
***45	-	10.20	7.71	5.47	16.70
46	33	19.30	7.75	6.18	16.78
47-55	-	11.60	7.75	6.18	16.78
56	34	12.90	7.92	6.18	20.91
57-63	-	12.81	7.92	6.18	20.91
64	35,36	15.51	8.24	6.43	20.71
65	37,38	15.25	9.14	7.66	22.32
66-70	-	15.05	9.14	7.66	22.32
71	39,40	15.44	9.37	8.05	22.57
72-74	-	15.28	9.37	8.05	22.57
75	41	15.29	9.57	8.52	23.37
76-80	-	15.35	9.57	8.52	23.37
81	42-45	23.80	10.61	8.68	27.88

* Downstream end of upper westerly portion of interceptor.
** Downstream end of upper easterly portion of interceptor.
*** Convergence point. (Cubic meters/second = 0.0283 cfs)

TABLE 11.

SUMMARY OF CONTRIBUTORY DISCHARGES TO MDC INTERCEPTOR: Method 2

Interceptor Component	Direct Tributary Areas	Present Interceptor Capacity (cfs)	Peak DWF (cfs)	Excess Infiltration (cfs)	Excess Inflow (cfs)
1	19-23	4.57	3.23	1.05	4.44
2-3	-	4.57	3.23	1.05	4.46
4	24	4.26	3.48	1.11	4.95
5	25	5.01	3.88	1.76	5.74
6	26,27	7.96	3.89	1.76	5.83
7-10	-	4.47	3.89	1.76	5.83
11	28,29	4.49	3.95	1.88	5.94
12-17	-	7.47	3.95	1.88	5.94
18	30-32	9.64	4.55	3.18	7.32
* 19	-	9.61	4.55	3.18	7.32
20-26	1-3	6.49	1.58	1.19	5.82
27	4	3.81	1.73	1.29	5.82
28	5,6	5.06	1.75	1.31	5.90
29	7	5.03	1.76	1.32	5.99
30	8,9	2.93	1.94	1.38	8.24
31	-	2.53	1.94	1.38	8.24
32	10,11	2.53	1.95	1.38	9.08
33	12,13	2.51	1.98	1.42	10.34
34	14	2.51	1.99	1.43	10.43
35	15-17	5.13	3.62	2.86	11.88
36-38	-	4.19	3.62	2.86	11.88
39	18	5.11	3.63	3.04	11.97
** 40-44	-	5.12	3.63	3.04	11.97
*** 45	-	10.20	8.18	6.22	19.29
46	33	19.30	8.22	7.92	19.54
47-55	-	11.60	8.22	7.92	19.54
56	34	12.90	8.62	7.92	20.32
57-63	-	12.81	8.62	7.92	20.32
64	35,36	15.51	8.94	8.17	20.12
65	37,38	15.25	9.45	9.03	21.01
66-70	-	15.05	9.45	9.03	21.01
71	39,40	15.44	10.60	11.20	23.05
72-74	-	15.28	10.60	11.20	23.05
75	41	15.29	10.84	11.56	24.01
76-80	-	15.35	10.84	11.56	24.01
81	42-45	23.80	11.95	11.72	34.37

* Downstream end of upper westerly portion of interceptor.
** Downstream end of upper easterly portion of interceptor.
*** Convergence point.

(Cubic meters/second = 0.0283 cfs)

TABLE 12.

SUMMARY OF CONTRIBUTORY DISCHARGES TO MDC INTERCEPTOR: METHOD 3

Interceptor Component	Direct Tributary Areas	Present Interceptor Capacity (cfs)	Peak DWF (cfs)	Excess Infiltration (cfs)	Excess Inflow (cfs)
1	19-23	4.57	4.93	1.23	4.24
2-3	-	4.57	4.94	1.23	4.24
4	24	4.26	5.31	1.31	4.80
5	25	5.01	5.75	2.08	5.55
6	26,27	7.96	5.76	2.08	5.64
7-10	-	4.47	5.76	2.08	5.64
11	28,29	4.49	5.84	2.28	5.77
12-17	-	7.47	5.84	2.28	5.77
18	30-32	9.64	6.62	3.56	6.96
*19	-	9.61	6.62	3.56	6.96
20-26	1-3	6.49	2.03	1.37	5.37
27	4	3.81	2.63	1.47	5.67
28	5,6	5.06	2.65	1.49	5.75
29	7	5.03	2.66	1.50	5.84
30	8,9	2.93	2.87	1.63	8.17
31	-	2.53	2.87	1.63	8.17
32	10,11	2.53	2.88	1.62	9.02
33	12,13	2.51	2.89	1.66	10.30
34	14	2.51	2.90	1.67	10.39
35	15-17	5.13	4.75	3.02	11.63
36-38	-	4.19	4.75	3.02	11.63
39	18	5.11	4.76	3.04	11.72
**40-44	-	5.12	4.76	3.04	11.72
***45	-	10.20	11.38	6.60	18.68
46	33	19.30	11.41	8.46	19.02
47-55	-	11.60	11.41	8.46	19.02
56	34	12.90	11.99	8.46	19.89
57-63	-	12.81	11.99	8.46	19.89
64	35,36	15.51	12.31	8.71	19.89
65	37,38	15.25	12.94	9.77	20.91
66-70	-	15.05	12.94	9.77	20.91
71	39,40	15.44	14.29	11.71	22.75
72-74	-	15.28	14.29	11.71	22.75
75	41	15.29	14.60	12.82	23.64
76-80	-	15.35	14.60	12.82	23.64
81	42-45	23.80	15.97	13.15	34.95

* Downstream end of upper westerly portion of interceptor.
** Downstream end of upper easterly portion of interceptor.
*** Convergence point.

(Cubic meters/second = 0.0283 cfs)

Cumulative tributary discharges estimated using the three stage/discharge methods are shown in Table 13 at several locations along the MDC interceptor for different flow conditions. Discharge estimates are provided at the downstream end of interceptor segments 19, 44, 45 and 81 for four different flow regimes, including: a) peak dwf (no rainfall/no wet weather induced high ground water); b) peak dwf plus infiltration (spring time high ground water snowmelt) ; c) peak dwf plus inflow (summer/fall rainfall periods); and, d) peak dwf plus infiltration and inflow (rainfall periods during spring snowmelt high ground water). The locations of the four segments are as follows: a) segment 19 is the downstream end of the upper westerly portion of the interceptor; b) segment 44 is the downstream end of the upper easterly portion of the interceptor; c) segment 45 is the confluent point; and d) segment 81 is the end of the interceptor.

The differences in estimated discharges resulting from using the three stage/discharge methods can be best seen by inspection of the relative ratios presented in Table 14. Discharge ratios at each segment are presented comparing method 2 with method 1, method 3 with method 1, and method 3 with method 2. With the exception of the upper westerly portion of the interceptor, flows computed by methods 2 and 3 are all increasingly greater than those derived from method 1. Overall system increases for methods 2 and 3 relative to method 1 range from 13% to 51%. The reversal noted for method 1 relative to method 2 at segment 19 is attributed to the fact that several least squares fitted stage/discharge curves (method 1) developed for key manholes 19-32 were above those developed for methods 2 and 3, resulting in greater discharge levels for the same stage (see Figure 12, page 38). This difference noted at segment 19 reverted to the general pattern of greater flows computed for method 3 relative to method 1 and can be explained by the fact that flows computed by method 3 will always exceed those computed by method 2, particularly for low flow conditions, and will converge at near full pipe conditions where the roughness factor in the variable roughness formulation with n_f = 0.015 (method 2) approaches the fixed roughness factor of n = 0.015 (method 3). Inspection of method 3 to method 2 discharge ratios in Table 14 indicate that these ratios steadily decreased starting from dwf conditions (low flow) to dwf & inflow & infiltration (high flow - near full pipe to full pipe conditions).

All infiltration estimates shown in Tables 10 through 14 derive from measurements conducted during spring snowmelt high ground water conditions. Over the course of the field program conducted during the prior study, significant clear water dry weather infiltration was observed in area 21. This area is tributary to the upper end of the westerly leg of the interceptor. Several thousand feet of trunk sewer pass through a golf course with continuously high ground water. Comparison of the sewage yield from this area (cfs/acre and cfs/mile of sewer) with other areas characterized by low infiltration yields indicated that the dry weather infiltration yields for this area are about 1.5, 0.75 and 1.75 cfs using the three respective stage/discharge methods. These dry weather infiltration estimates have been reported in Tables 10 through 14 as peak dwf.

TABLE 13.

OVERALL COMPARISON OF DISCHARGE FOR DIFFERENT FLOW CONDITIONS COMPUTED BY THREE STAGE/DISCHARGE METHODS

Discharge (cfs)

	DWF			DWF & Infiltration			DWF & Inflow			DWF & Infiltration & Inflow		
Segment	*	**	***	*	**	***	*	**	***	*	**	***
19	4.77	4.55	6.62	8.58	7.73	10.18	13.06	11.87	13.58	16.87	15.05	17.14
44	2.94	3.63	4.76	4.60	6.67	7.80	11.35	15.60	16.48	13.01	18.64	19.52
45	7.71	8.18	11.38	13.18	14.40	17.98	24.41	27.47	30.06	29.88	33.69	36.66
81	10.61	11.95	15.97	19.29	23.67	29.09	38.49	46.32	50.92	47.17	58.04	64.04

* Method 1 - Least squares (wls) estimate of $\hat{S}$, variable n (n_f = 0.015).
** Method 2 - Plan & profile slope, variable n (n_f = 0.015).
*** Method 3 - Plan & profile slope, fixed n = 0.015.

(Cubic meters/second = 0.0283 cfs)

TABLE 14.

RELATIVE RATIOS OF DISCHARGE FOR DIFFERENT FLOW CONDITIONS COMPUTED BY THREE STAGE/DISCHARGE METHODS

Segment	DWF			DWF & Infiltration			DWF & Inflow			DWF & Infiltration & Inflow		
	*	**	***	*	**	***	*	**	***	*	**	***
19	.95	1.39	1.46	.90	1.19	1.32	.91	1.04	1.14	.89	1.02	1.14
44	1.24	1.62	1.31	1.45	1.70	1.17	1.37	1.45	1.06	1.43	1.50	1.05
45	1.06	1.48	1.39	1.09	1.23	1.25	1.13	1.23	1.09	1.13	1.23	1.09
81	1.13	1.51	1.34	1.23	1.51	1.23	1.20	1.32	1.10	1.23	1.36	1.10

* Ratio of Discharge - Method 2: Method 1.
** Ratio of Discharge - Method 3: Method 1.
*** Ratio of Discharge - Method 3: Method 2.

Method 1: least squares (wls) estimate of $\hat{S}$, variable n (n_f = 0.015).
Method 2: Plan & profile slope, variable n (n_f = 0.015)
Method 3: Plan & profile slope, fixed n = 0.015.

SECTION 7

ALTERNATIVE REHABILITATION PROGRAM COSTS

7.1 Foreword

Details of three alternative cost-effective infiltration/inflow correction control programs to minimize backwater effects in areas tributary to the MDC interceptor are presented in Section 7.2. The three rehabilitation programs were developed on the basis of the three different sets of tributary discharges derived from the various stage/discharge methods presented in Section 6. Cost estimates for the three programs are provided in Section 7.3. A discussion of measurement program costs versus the cost of erroneous rehabilitation recommendation is presented in Section 7.4.

7.2 Alternative Infiltration/Inflow Correction Programs

The primary objective for the control of infiltration/inflow conditions in the prior study was to minimize the occurrences of surcharge and sewer backup along the MDC interceptor system. The existing MDC program entails construction of a complete new replacement interceptor system parallel to the existing lines, costing $4.6 million (1975). The alternative plans considered consist of upgrading the hydraulic capacity of a limited number of interceptor segments and minimizing extraneous water to the limit considered practical and cost-effective. The costs for construction of new segments to increase hydraulic conveyance, the O&M sewage treatment cost of extraneous flows, and inflow and infiltration rehabilitation costs were all considered in deriving the minimum cost program for each set of dry and wet weather flows calculated from the three alternative stage/discharge procedures.

The technical criteria used in the prior infiltration/inflow study for delineating new interceptor segment construction and infiltration/inflow rehabilitation programs entailed the maintenance of at least 1 cfs dwf reserve in the upper reaches of the interceptor system (segments 1 through 19 and segments 20 through 44) and at least 4 cfs dwf reserve in the main downstream system (segments 45 through 81). In addition, full pipe open-channel flow with no surcharge in the interceptor was adopted for peak wet weather conditions of concurrent infiltration and inflow. A slight wet weather reserve was also desired to allow for nominal infiltration along the interceptor. It was assumed in the analysis that the hydraulic capacity of the existing interceptor segments requiring relief would not be considered and parallel segments would be constructed with at least the next larger commercial size.

The flow reduction effectiveness of infiltration inflow rehabilitation correction for any subarea is set at 85%. This reduction is high considering recent evidence of lower rehabilitation actually achieved in practice. The 85% level was assumed in earlier studies and maintained in this study.

Overview details of the three control programs for each set of desired flows are presented in Tables 15, 16 and 17. Information presented in the tables are as follows: a) interceptor component numbers; b) tributary areas; c) and d) keys to denote whether infiltration and/or inflow correction is envisioned per area; e) cumulative wet weather flow (cfs) during concurrent infiltration and inflow periods resulting from infiltration and/or inflow controls; f) existing capacity (cfs) of the interceptor segments if multiple segments, then limiting or smallest capacity); g) key to denote whether capacity expansion necessary, and if so, hydraulic capacity (cfs) of new segments (if multiple segments, then limiting or smallest capacity of new segments); h) key to denote the reason for expansion; i) excess capacity (cfs) during peak dwf for modified interceptor system; and, j) excess capacity (cfs) during peak wet weather conditions for modified interceptor system.

The control program presented in Table 15 for the first method of flow estimation considers the following elements: a) infiltration rehabilitation in areas 19-23, 26-32, 33, and 35-38; b) inflow remedial programs in all areas except for 42-45; and, c) new interceptor segments in components 27, and 31 through 45. The interceptor construction program for this alternative consists of 3365 feet of 15 to 30 inch sewer. The peak wet weather discharge of segment 81 equals 20.85 cfs and the peak dwf, infiltration and inflow contributions are 9.33 cfs, 3.5 cfs and 8.02 cfs, respectively. With reference to Table 10, overall system infiltration and inflow reductions equal 60 and 71% respectively. Peak dwf is reduced by 12% by the reduction of dry weather infiltration in area 21.

The control program presented in Table 16 for the second method of flow estimation consists of infiltration rehabilitation in areas 1-42, inflow remedial programs in all areas and the addition of interceptor relief sewers in components 30 through 45. The interceptor construction program for this alternative consists of 3106 feet of 21-30 inch sewer. The peak wet weather discharge at segment 81 equals 18 cfs and the peak dwf, infiltration and inflow contributions are 11.10 cfs, 1.74 cfs and 5.16 cfs, respectively.

The control program presented in Table 17 for the third method of flow estimation entails complete infiltration and inflow rehabilitation in all areas and the addition of interceptor relief sewers in segments 4,5, 7-11, 31-45, 47-63, and 65-80. The interceptor construction program consists of 4847 feet of 15-30 inch and 8271 feet of 36-39 inch sewer. The peak wet weather discharge equals 21.69 cfs and the peak dwf, infiltration and inflow contributions are 14.81 cfs, 1.97 cfs and 5.24 cfs, respectively. The differences between programs 1 and 2 lie in the degree of overall infiltration control, inflow control in areas 42-45, and relief sewer construction along component 27. Roughly 260 feet of new 15 inch sewer is required

TABLE 15.
OVERVIEW OF REHABILITATION PROGRAM - METHOD 1

Interceptor Component	Tributary Area	Infiltration Control*	Inflow Control*	Discharge (cfs) Total Wet Flow***	Present Capacity†	Capacity Expansion††	Reason†††	DWF Reserve	Wet Reserve
1	19-23	y**	y	3.66	4.57	N	-	2.10	.91
2-3	-	-	-	3.66	4.57	N	-	2.10	.91
4	24	y	y	3.77	4.26	N	-	1.58	.49
5	25	y	y	4.29	5.01	N	-	2.11	.72
6	26, 27	y	y	4.31	7.96	N	-	5.05	3.65
7-10	-	-	-	4.31	4.47	N	-	3.82	.06
11	28, 29	y	y	4.44	4.49	N	-	3.78	.05
12-17	-	-	-	4.44	7.47	N	-	4.49	3.03
18	30-32	y	y	5.29	9.64	N	-	6.14	4.35
19	-	-	-	5.29	9.61	N	-	6.11	4.32
20-26	1-3	N	y	3.89	6.49	N	-	4.61	2.60
27	4	y	y	4.01	3.81	6.91	2	4.93	2.90
28	5, 6	y	y	4.05	5.06	N	-	3.06	1.01
29	7	y	y	4.07	5.03	N	-	3.02	.96
30	8, 9	y	y	4.20	2.93	6.16	2	4.11	1.96
31	-	-	-	4.20	2.53	6.14	2	4.09	1.94
32	10, 11	N	y	4.30	2.53	6.14	2	4.08	1.84
33	12, 13	N	y	4.43	2.51	6.13	1, 2	4.03	1.70
34	14	N	y	4.46	2.51	6.13	1, 2	3.93	1.67
35	15-17	N	y	5.59	5.13	7.80	2	4.87	2.21
36-38	-	-	-	5.59	4.19	6.82	2	3.89	1.23
39	18	N	y	5.64	5.11	8.29	2	5.35	2.65
40-44	-	-	-	5.64	5.12	8.30	2	5.36	2.66
45	-	-	-	10.93	10.20	14.96	2	8.52	4.03
46	33	y	y	11.09	19.30	N	-	12.86	8.23
47-55	-	-	-	11.09	11.60	N	-	5.12	.59
56	34	y	y	11.88	12.90	N	-	6.25	1.02
57-63	-	-	-	11.88	12.81	N	-	6.16	.93
64	35, 36	y	y	12.21	15.51	N	-	8.54	3.30
65	37, 38	y	y	13.52	15.25	N	-	7.39	1.73
66-70	-	-	-	13.52	15.05	N	-	7.19	1.53
71	39, 40	N	y	14.18	15.44	N	-	7.35	1.26
72-74	-	-	-	14.18	15.28	N	-	7.19	1.10
75	41	N	y	14.97	15.29	N	-	6.99	.32
76-80	-	-	-	14.97	15.35	N	-	7.06	.38
81	42-45	N	N	20.67	23.80	N	-	14.47	3.13

* Infiltration/Inflow Control: y - Control @ 85% Effectiveness. N - No Control.

** 1.5 cfs dry weather infiltration (Area 1), controlled at 85% effectiveness.

*** Total flow after control during concurrent infiltration/inflow condition.

† Minimum Hydraulic Capacity.

†† "N" implies no additional capacity need, otherwise discharge given at full pipe capacity of new replacement segment

††† "1" implies needed for DWF conditions, "2" implies capacity expansion for wet weather conditions.

1 cubic meter/second = 0.0283 cfs

TABLE 16.
OVERVIEW OF REHABILITATION PROGRAM - METHOD 2

Discharge (cfs)

Interceptor Component	Tributary Area	Infiltration Control*	Inflow Control*	Total Wet Flow***	Present Capacity†	Capacity Expansion††	Reason†††	DWF Reserve	Wet Reserve
1	19-23	y**	y	3.21	4.57	N	-	2.18	1.36
2-3	-	-	-	3.21	4.57	N	-	2.18	1.36
4	24	y	y	3.55	4.26	N	-	1.63	.71
5	25	y	y	4.16	5.01	N	-	1.98	.85
6	26,27	y	y	4.18	7.96	N	-	4.91	3.78
7-10	-	-	-	4.18	4.47	N	-	1.43	.29
11	28,29	y	y	4.28	4.49	N	-	1.39	.21
12-17	-	-	-	4.28	7.47	N	-	4.37	3.19
18	30-32	y	y	5.28	9.64	N	-	5.94	4.36
19	-	-	-	5.28	9.61	N	-	5.91	4.23
20	1-3	y	y	2.63	6.49	N	-	4.91	3.86
27	4	y	y	2.80	3.81	N	-	2.08	1.01
28	5-6	y	y	2.83	5.06	N	-	3.28	2.23
29	7	y	y	2.86	5.03	N	-	3.27	2.17
30	8,9	y	y	3.38	2.93	6.16	2	4.23	2.78
31	-	-	-	3.38	2.53	6.14	2	4.21	2.76
32	10,11	y	y	3.51	2.53	6.14	2	4.20	2.63
33	12,13	y	y	3.74	2.51	6.13	1,2	4.15	2.39
34	14	y	y	3.77	2.51	6.13	1,2	4.14	2.36
35	15-17	y	y	5.84	5.13	7.80	2	4.18	1.96
36-38	-	-	-	5.84	4.19	6.82	2	3.20	0.98
39	18	y	y	5.88	5.11	8.29	2	4.66	2.41
40-44	-	-	-	5.88	5.12	8.30	2	4.67	2.42
45	-	-	-	11.16	10.20	14.96	2	7.63	3.80
46	33	y	y	11.49	19.30	N	-	11.93	7.81
47-55	-	-	-	11.49	11.60	N	-	4.23	.13
56	34	y	y	12.01	12.90	N	-	5.12	.89
57-63	-	-	-	12.01	12.81	N	-	5.04	.80
64	35,36	y	y	12.33	15.51	N	-	7.42	3.18
65	37,38	y	y	13.11	15.25	N	-	6.66	2.14
66-70	-	-	-	13.11	15.05	N	-	6.45	1.94
71	39-40	y	y	14.89	15.44	N	-	5.69	.55
72-74	-	-	-	14.89	15.28	N	-	5.53	.39
75	41	y	y	15.33	15.29	N	-	5.30	-
76-80	-	-	-	15.33	15.35	N	-	5.36	.02
81	42-45	N	y	18.00	23.80	N	-	12.70	5.80

1 cubic meter/second = 0.0283 cfs

* Infiltration/Inflow Control: y - Control @ 85% Effectiveness. N - No Control.

** 1.0 cfs dry weather infiltration (Area 21), controlled at 85% effectiveness.

*** Total flow after control during concurrent infiltration/inflow condition.

† Minimum Hydraulic Capacity.

†† "N" implies no additional capacity need, otherwise discharge given at full pipe capacity of new replacement segment

††† "1" implies needed for DWF conditions, "2" implies capacity expansion for wet weather conditions.

TABLE 17.
OVERVIEW OF REHABILITATION PROGRAM - METHOD 3

Interceptor Component	Tributary Area	Infiltration Control*	Inflow Control*	Discharge (cfs) Total Wet Flow***	Present Capacity†	Capacity Expansion††	Reason†††	DWF Reserve	Wet Reserve
1	15-23	y**	y	4.09	4.57	N	-	1.49	.48
2-3	-	-	-	4.09	4.57	N	-	1.49	.48
4	24	y	y	4.74	4.26	6.43	1,2	2.61	2.17
5	25	y	y	5.41	5.01	7.53	1,2	3.27	2.12
6	26,27	y	y	5.43	7.96	N	-	3.69	2.53
7-10	-	-	-	5.43	4.47	6.73	1,2	2.46	1.30
11	28,29	y	y	5.56	4.49	6.73	1,2	2.41	1.20
12-70	-	-	-	5.56	7.47	N	-	3.12	1.91
18	30-32	y	y	6.71	9.64	N	-	4.51	2.93
19	-	-	-	6.71	9.61	N	-	4.48	2.90
20-26	1-3	y	y	3.04	6.49	N	-	4.46	3.45
27	4	y	y	3.70	3.81	N	-	1.18	.11
28	5,6	y	y	3.73	5.06	N	-	2.41	1.33
20	7	y	y	3.76	5.03	N	-	2.38	1.27
30	8,9	y	y	4.34	2.93	6.16	2	3.30	1.82
31	-	-	-	4.34	2.53	6.14	1,2	3.28	1.80
32	10,11	y	y	4.47	2.53	6.14	1,2	3.27	1.67
33	12,13	y	y	4.69	2.51	6.13	2	3.24	1.44
34	14	y	y	4.71	2.51	6.13	2	3.23	1.42
35	15-17	y	y	6.95	5.13	7.80	2	3.05	.85
36-38	-	-	-	6.95	4.19	6.82	2	2.07	-
39	18	y	y	6.98	5.11	8.29	2	3.53	1.31
40-44	-	-	-	6.98	5.12	8.30	2	3.54	1.32
45	-	-	-	13.68	10.20	14.96	2	5.07	1.28
46	33	y	y	14.04	19.03	N	-	9.38	5.26
47-55	-	-	-	14.04	11.60	15.86	1,2	5.94	1.82
56	34	y	y	14.75	12.90	17.38	2	6.87	2.63
57-63	-	-	-	14.75	12.81	17.38	2	6.87	2.63
64	35-36	y	y	15.11	15.51	N	-	4.69	.40
65	37,38	y	y	16.06	15.24	17.38	2	5.93	1.32
66-70	-	-	-	16.06	15.05	17.38	2	5.93	1.32
71	39,40	y	y	17.97	15.44	22.68	2	9.88	4.71
72-74	-	-	-	17.97	15.28	21.51	2	8.71	3.54
75	41	y	y	18.58	15.29	21.51	2	8.40	2.93
76-80	-	-	-	18.58	15.35	22.68	2	9.57	4.10
81	42-45	y	y	21.69	23.80	N	-	9.32	2.11

1 cubic meter/second = 0.0283 cfs

* Infiltration/Inflow Control: y - Control @ 85% Effectiveness.
N - No Control.

** 1.75 cfs dry weather infiltration (Area 21), controlled at 85% Effectiveness.

*** Total flow after control during concurrent infiltration/inflow condition.

† Mimimum Hydraulic Capacity.

†† "N" implies no additional capacity need, otherwise discharge given at full pipe capacity of new replacement segment

††† "1" implies needed for DWF conditions, "2" implies capacity expansion for wet weather conditions.

in program 1 for segment 27. The critical decision in developing the least cost mix of controls for method 2 is relief capacity for the 34" interceptor sewer along segments 75-80. Inspection of Tables 15 and 16 indicated that the wet weather reserve is adequately met in Plan 1 and just barely met in Plan 2 for these segments. If the reduced infiltration program envisioned for method 1 were used for the method 2 relief plan, then much more costly relief sewers along these segments would be necessary.

The differences between the plans for methods 2 and 3 lie principally in the degree of relief sewer construction. Flows estimated using method 3 are greater in all portions of the interceptor system requiring major relief in both of the upper westerly and easterly portions of the interceptor as well as along the main stem.

7.3 Costs of Infiltration/Inflow Correction Programs

In this section, the overall correction program costs are presented for the three programs associated with each method of flow estimation. Estimates include infiltration rehabilitation, inflow remedial repairs, construction of new relief segments and O&M treatment costs of extraneous flow.

Infiltration rehabilitation programs for each tributary area were developed in the following manner. A surficial geology map and topographic maps were used to estimate the fraction of each area covered per soil type and the fraction of each soil type considered steep or gently sloped. These details are reported elsewhere (1). This information was used to estimate the percentage of pipes with potential leakage within each catchment area. A summary of the length of pipe with potential leakage problems per tributary area considered in the infiltration rehabilitation programs for each method of flow estimation is presented in Table 18. A weighted unit cost of $6.50/ linear foot (ENR = 2960) was used to account for testing and sealing leaky pipe joints, replacement of pipe, manhole grouting and repair. Engineering, legal and administrative costs equal 25%. The estimate sewer system evaluation study for infiltration control is estimated to be $278,300 for the entire study area. The overall infiltration rehabilitation program costs for the three alternative methods of discharge computation are $1,008,300, $1,533,800 and $1,607,800. These estimates assume that the evaluation survey costs for portions of the area are proportional to overall study costs.

Inflow remedial program costs were developed through detailed costing of remedial actions noted by field inspection of representative areas with known high, moderate and slight inflow conditions. These costs were then used to estimate inflow correction programs in all areas tributary to the MDC interceptor. The inflow remedial program cost for the entire area, including a 25% allowance for engineering, legal and administrative costs equals $1,138,200 (ENR = 2960). The inflow sewer system evaluation study for the entire area is estimated to cost $100,000. The flow remedial program cost for areas 42-45, including engineering, legal and administration equals $90,800. The overall costs of the inflow remedial correction programs for the three methods of discharge computation are $1,147,400, $1,238,200 and $1,238,200.

TABLE 18
INFILTRATION REHABILITATION PROGRAM DETAILS

Interceptor Component	Direct Tributary Areas	Length of Pipe With Potential Leakage Problems (1000 ft)	Method 1*	Method 2*	Method 3*
1	19-23	48.8	y	y	y
2-3	-	-			
4	24	1.2	-	-	y
5	25	10.	y	y	y
6	26,27	0.2	y	y	y
7-10	-	-			
11	28,29	2.6	y	y	y
12-17	-	-			
18	30-32	13.1	y	y	y
19	-	-			
20-26	1-3	27.4	-	y	y
27	4	1.2	-	y	y
28	5,6	0.4	-	y	y
29	7	0.7	-	y	y
30	8,9	0.5	-	y	y
31	-	-	-	y	y
32	10,11	0.6	-	y	y
33	12,13	1.3	-	y	y
34	14	0.3	-	y	y
35	15-17	9.3	-	y	y
36-38	-	-			
39	18	0.3	-	y	y
40-44	-	-			
45	-	-			
46	33	2.7	y	y	y
47-55	-	-			
56	34	2.8	-	y	y
57-63	-	-			
64	35,36	0.4	y	y	y
65	37,38	23.2	y	y	y
66-70	-	-			
71	34-40	4.2	-	y	y
72-74	-	-			
75	41	4.3	-	y	y
76-80	-	-			
81	42-45	7.5	-	-	y
	Subtotal	163.0	101.0	155.5	163.

Pipe Footage Requiring Rehabilitation (1000 ft) (1m = 3.28 ft)

* y - Denotes inclusion of area(s) for infiltration and rehabilitation.

Pipe footage and unit costs for the relief sewer expansion programs prepared for each of the three discharge estimation methods are shown in Table 19. Construction costs for the three alternative programs, including engineering, legal and administration (25%) equal $584,900, $560,300 and $3,032,000 (ENR =2960). The cost of a complete new interceptor replacement system is estimated to cost $6,670,000 (ENR = 2960).

Present value costs for treatment of extraneous wet weather discharges for the various plans considered are presented in Table 20. Inflow & infiltration estimates for each plan prepared on the basis of the three methods of discharge estimation are presented in Table 20-I. The "no control" program estimates are presented to reflect extraneous wet weather discharge levels for the plan consisting of the construction of a complete new interceptor system. Present value O&M treatment costs are presented in Table 20-II for the various cases of residual inflow and infiltration. The present value treatment cost of $879/mgd is for primary treatment at the Nut Island Treatment Facility in Boston and is based on a 1980 annual O&M cost of $80/mgd and an interest rate of 6-3/4% for a 20-year discount period. Total present value treatment costs for the remaining extraneous discharges for the three infiltration/inflow correction programs equal $423,300, $228,600 and $248,600, respectively. The treatment costs of extraneous wet weather discharge for the plan based on flow estimation method 1 is roughly $200,000 higher than the costs for the other two methods.

One interesting sidepoint that does not enter the cost comparison of the overall infiltraton/inflow correction programs resulting from using the three different methods of flow estimation is the dry weather sewage treatment costs. The total dry weather sewage discharge after control of dry weather infiltration in area 21 for the three methods of discharge estimation are 9.33 cfs, 11.10 cfs and 11.48 cfs, respectively. Present value treatment costs for these three flow estimates equal $4,609,800, $5,486,300 and $7,154,900. These differences could reflect possible inequities in computing user charges for the various communities within the study area.

A comparison of the overall present value of infiltration/inflow correction program costs prepared for each method of flow estimation is presented in Table 21-I. Total present value costs of the control program prepared using the first method of discharge estimation is $417,300 and $2,963,000 less than the equivalent control programs derived from using the second and third method of estimation. Present value costs for the alternative construction of a complete new relief interceptor system are presented in Table 21-II for comparison. It is assumed that the complete new interceptor replacement system construction costs would be unaffected by the choice of flow estimation. Design details used in developing the complete new interceptor program are not available.

7.4 Discussion of Results

Three different least cost control programs have been prepared as alternatives to a program consisting of complete replacement of the interceptor system. The three programs were based on three different methods for computing discharge levels from stage information collected during field pro-

TABLE 19.
RELIEF SEWER CONSTRUCTION PROGRAMS

Interceptor Segment	Existing Size/Shape (in.)	New Relief Size/Shape (in.)	Segments Footage (ft)	Plan for Method 1*	Plan for Method 2*	Plan for Method 3*
4	18 c	21 c	358	-	-	y
5	18 c	21 c	239	-	-	y
8-10	18 c	21 c	879	-	-	y
11	18 c	21 c	265	-	-	y
27	12 c	15 c	259	y	-	-
30-34	15 c	21 c	1353	y	y	y
35	20x20.5 ov	24 c	243	y	y	y
37-38	20x20.5 ov	24 c	572	y	y	y
39-42	20x20.5 ov	24 c	927	y	y	y
45	26x26 3/4 ov	30 c	11	y	y	y
47-61	32x32 3/4 ov	36 c	6031	-	-	y
66-77	34x36 ov	39 c	2240	-	-	y
		Subtotals		3365 ft	3106 ft	13,118 ft

c - circle
ov- ovoid
* y - denotes that relief sewer construction is necessary.

Relief sewer construction unit costs ($/lineal foot) including manholes, tyins, engineering, legal and administration are as follows:

15 in. - 95 30 in. - 500
21 in. -168 36 in. - 254
24 in. -188 39 in. - 289

1 in. = 2.54 cm
1 m = 3.28 ft

TABLE 20.
PRESENT VALUE TREATMENT COSTS FOR EXTRANEOUS
WET WEATHER DISCHARGES

I. Wet Weather Infiltration and Inflow Discharges (cfs):

	Method 1		Method 2		Method 3	
	A*	B**	A*	B**	A*	B**
No Control	27.88	8.68	34.37	11.72	34.95	13.15
Plan Developed for Method 1 Discharges	8.22	3.50				
Plan Developed for Method 2 Discharges	-	-	5.16	1.74	-	-
Plan Developed for Method 3 Discharges	-	-	-	-	5.24	1.97

* Inflow (cfs)
** Infiltration (cfs)

(Cubic meters/second = 0.0282 cfs)

II. Present Value Treatment Costs ($1,000):
(6.75% @ 20 years)

	Method 1		Method 2		Method 3	
	A*	B**	A*	B**	A*	B**
No Control	471.8	704.9	581.6	951.9	591.4	1,068.0
Plan Developed for Method 1 Discharges	139.0	284.3	-	-	-	-
Plan Developed for Method 2 Discharges	-	-	87.3	141.3	-	-
Plan Developed for Method 3 Discharges	-	-	-	-	88.6	160.0

*A- Inflow treatment costs based on 50 rainfall occurrences per year lasting six hours.

**B-Infiltration treatment costs based on 60 days/year.
Costs based on 1980 annual cost of primary treatment equal to $80/million gallons. Present value cost = $879/million gallons (6-3/4%, 20 years).

TABLE 21.
COMPARISON OF OVERALL INFILTRATION/INFLOW
CORRECTION COST PROGRAMS

	Stage/Discharge Methods Cost ($1,000)		
I. Cost-Effective Corrective Programs	1	2	3
Infiltration Control	1,008.3	1,533.8	1,607.8
Inflow Control	1,147.4	1,238.2	1,238.2
Increased Hydraulic Conveyance of Selected Segments	584.9	560.3	3,032.0
Treatment of Residual Infiltration/Inflow	423.0	228.6	248.6
	3,163.6	3,580.9	6,126.6
II. Complete New Interceptor System	1	2	3
Construction	6,670.0	6,670.0	6,670.0
Treatment of Infiltration/Inflow	1,176.7	1,533.5	1,569.4
	7,846.7	8,203.5	8,329.4

(ENR = 2960)

grams. Each program satisfied the imposed technical criteria for mitigating backwater and surcharge conditions. The overall mix of recommendations and their associated costs are, however, different. These differences reflect erroneous recommendations for decision-making and potential inefficient expenditures of resources. An overview of the general differences for the three plans is presented in Table 22.

There is an associated cost involved in preparing the stage/discharge relationships using field discharge calibration points. The informational value and cost of this effort relative to the cost of erroneous recommendations deriving from a methodology(s) in which the effort is not performed can be roughly estimated as follows. It is estimated that the field engineering cost for obtaining direct discharge measurements together with in-house office engineering cost for computing the least squares fit of the resulting data should be no more than $200 to $500 per key manhole station.* Since this effort is normally not performed, this cost should be considered as additive to the routine engineering cost of Step 1, infiltration/inflow studies. Assuming that the unit cost for gathering and processing this information was $300 per key manhole for the 45 areas, then the implied information gathering cost would be $13,500. This estimate of resources spent can be compared with the additional and erroneous cost differences for the control plan recommendations using methods 2 and 3. The cost differences between plans prepared using methods 2 and 3 in comparison to method 1 are $417,300 and $2,963,000. The net value of method 1 results in a cost savings of $417,300 - $13,500 = $403,800 and $2,963,000 - $13,500 = $2,949,500, respectively. It is also interesting to note the control plan mix and cost differences associated with methods 2 and 3. Simple inclusion of the variable roughness formulation instead of the usual practice of using fixed n values in Manning's equations results in a substantially different overall control plan. The cost difference between the two methods equals roughly 2.5 million dollars.

The benefit of carefully preparing stage/discharge rating curves in infiltration/inflow studies by performing primary measurements and making the best use of this information through least squares procedures is significant. The insights gained through a comparative assessment of stage/discharge procedures used for infiltration/inflow studies applies equally as well to any sewerage system management study.

---

* The field effort represented roughly 90% of the additional costs. The cost of office calculations associated with the least squares computations were minor. Most of the office effort is devoted to obtaining estimates of hydraulic parameters which are required for any procedure.

TABLE 22
OVERVIEW OF CONTROL PLANS DEVELOPED USING DIFFERENT METHODS OF FLOW ESTIMATION

Flow Estimation Methodology	Method 1	Method 2	Method 3
1 (Least squares estimation of slope in Manning's Equation with variable roughness factor coefficient)	∘ infiltration control - 60% ∘ inflow control - 71% ∘ new relief segments in portions of upper westerly interceptor system.	-	-
2 (Manning's Equation with plan & profile slope and variable roughness coefficient)	-	∘ infiltration control - 85% ∘ inflow control - 85% ∘ new relief segments in portions of upper westerly interceptor system (about the same as plan 1)	-
3 (Manning's Equation with plan & profile slope and fixed roughness coefficient	-	-	∘ infiltration control - 85% ∘ inflow control - 85% ∘ new relief segments throughout the entire system
Overall Program Costs	$3,163,600	$3,580,900	$6,126,660

REFERENCES

1. Environmental Design & Planning, Inc., Infiltration/Inflow Study for West Roxbury-Brookline-Newton-Dedham Area. Metropolitan District Commission.

2. Pisano, W.C., et al; Dry Weather Deposition and Flushing for Combined Sewer Overflow Pollution Control. EPA 600/2-79-133, U.S. EPA, August, 1979.

3. Wilcox, E.R., A Comparative Test of the Flow of Water in 8-Inch Concrete and Vitrified Clay Sewer Pipe. University of Washington, Exp. Sta. Series Bull. 27, 1924.

4. Yarnell, D.L. and S.M. Woodward, The Flow of Water in Drain Tile. Dept. of Agriculture Bull. No. 854, U.S. Govt. Printing Office, Washington, D.C., 1920.

5. Johnson, C.F., Determination of Kutter's n for Sewers Partly Filled. Transactions, ASCE Journal, Vol. 109, 1944.

6. Camp, T.R., Discussion - Determination of Kutter's n for Sewers Partly Filled, by C.F. Johnson. Transactions, ASCE Journal, Vol. 109, 1944.